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Storm King Highway, New York

By H. Eltinge Breed*

SYNOPSIS—The contract for Storm King Road, on the west side of the Hudson River, has at last been let. The article sketches the history of various plans to build this road, gives extraordinary survey methods employed and present plans for the work; extracts from unusual specifications.

The projected Storm King Road lies along the famous gateway to the Highlands on the west bank of the Hudson River about 50 mi. from New York City. For scenic grandeur no section of the Eastern country surpasses these mountains between West Point and Cornwall, with their rugged slopes and constantly changing vistas. Many of them rise abruptly from the river to an elevation of 1,400 ft., with sides so precipitous as to have sheer drops, sometimes of 500 ft.

The road along the river across the hitherto impassable faces of these mountains will be the connecting link between northern New York and the great new playground of New York and New Jersey, the Palisades Interstate Park. It will also provide an outlet from Highland Falls, West Point and other towns below to Albany and intermediate places along the river. Formerly, traffic from these towns had to traverse a dangerous mountain pass some distance back from the river in order to reach the Albany road. The New York State Highway Commission has designated the West Point-Cornwall Road as No. 5,498; the section across Storm King Mountain will be part of Route No. 3 of the state highway system, extending from New Jersey to Albany.

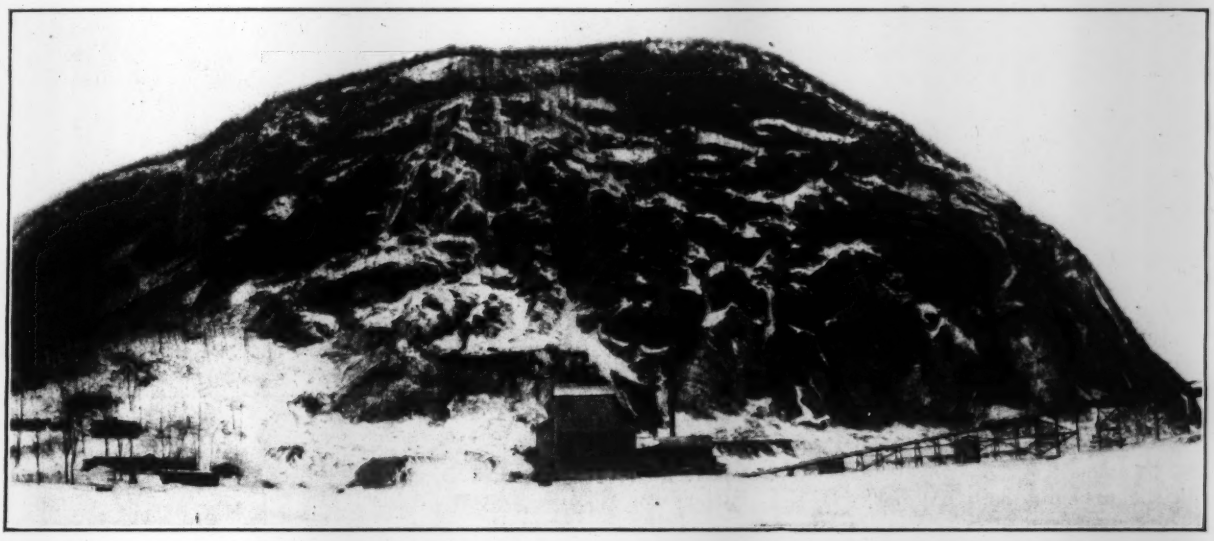


FIG. 1. STORM KING MOUNTAIN, ON THE WEST SHORE OF THE HUDSON RIVER, NEW YORK, ON THE FACE OF WHICH IT IS PROPOSED TO BUILD A STATE HIGHWAY

The top of the mountain is 1,340 ft. above the river. The proposed highway passes around the nose at an elevation of 290 ft. above the river

Earlier Surveys and Plans for Highway

For this important piece of work there have been many surveys by different engineers and many designs and redesigns. The original survey was made in 1904, almost wholly by the stadia method, and the plan was drawn to show 2-ft. contours when possible, and in the steep sections, 10-ft. contours. The maximum grade designed in these plans was 5%.

Along the side of Storm King Mountain a sheer rock face rises 360 ft. above the river, and close examination shows that this is shattered with seams running in every conceivable direction. As the 5% grade would have brought the road profile to an elevation of only 200 ft., instead of 360 ft., it would have been necessary, according to this first survey, to tunnel 700 ft. through the dangerous and uncertain rock on the face of the mountain. These original plans also called for 1,000 ft. of gallery construction; the rest of the road was to be located along the face of the mountain.

*First Deputy Commissioner of Highways, Albany, N. Y.

FIG. 2. CROW'S NEST MOUNTAIN, BELOW STORM KING, AROUND THE BASE OF WHICH THE PROPOSED HIGHWAY IS TO RUN

Plans were completed for the work covering a distance of 2.31 mi., and the contract was awarded Dec. 2, 1907, for $187,000. The contractor had barely commenced construction when he was stopped by a taxpayer's injunction brought to protect the county from the exorbitant demands made by a local stone company for right-of-way. This controversy lasted for over eight years.

It was finally adjusted, and in 1913 a new survey was made. This time the road was so designed that the limiting grade was 7%. This brought the road to an elevation of nearly 390 ft. above the river, around the face of Storm King, and obviated the necessity for any tunnel construction. Again, in 1914, bids were received. Of these, the lowest, for only 1.27 mi. of the road, was $197,000.

Because of the high cost of this one section, it became necessary to reject all bids and to redesign the road; otherwise the amount of money available would not have been sufficient to build both sections. If the contract for this first section had been let and completed it would have stood isolated with no outlet to the south unless other funds had been appropriated by special legislative enactment.

Redesigning and the Present Plans

With further redesigning it was found that the whole road could be built within the appropriation, and on July 26, 1915, bids were received for this work, the contract being awarded to the John L. Hayes Construction Co. of Yonkers, N. Y., for $273,736.70. In the redesign two short bridges along the south side of Storm King Mountain, which were a part of the 1913 design, were eliminated because in addition to being a source of constant expense for maintenance there was always the possibility that some of the overhanging rock might become loosened and fall on them. At these points the proposed alignment has been carried back into ledge rock so that there the roadway is supported on a shelf cut into solid rock.

A concrete viaduct was also eliminated. This had been planned to cross at the top of a slide, which appeared to be in motion. Subsequent field examination discovered that directly under the loose material of the slide there was a fairly sound talus. The center line was therefore thrown back so as to make the road over this section by excavation, which effected the saving of a considerable amount of money as well as the elimination of a curve of 250-ft. radius.

Extraordinary Survey Methods Employed

In the many surveys made, it was necessary quite often to devise extraordinary means of carrying on the work because of the rough, dangerous and impassable slopes. The first survey was made in the winter, when

FIG. 3. NOSE OF STORM KING MOUNTAIN, SHOWING TRACKS OF WEST SHORE R.R.

FIGS. 4 AND 5. TYPICAL BOULDER SLOPES AT HIGHWAY CENTER LINE

the snow and ice made every foothold precarious. For protection against slipping, all of the men wore creepers, as well as belts which were attached to ropes and snubbed around a convenient tree, so that should the wearer lose his foothold he would be held by the rope instead of dropping to the railroad tracks. Even with these precautions two or three accidents occurred which only good luck kept from being very serious.

A part of Storm King is absolutely inaccessible because of the almost perpendicular drop on the face of the mountain and on the first survey the topography was worked out by triangulation from such points as stood out and were readily distinguishable from two instruments, as well as such points as could be marked. Observations were taken from a base along the West Shore railroad tracks on which two transits were used simultaneously. As there were still many places that were inaccessible, an ordnance sergeant in the United States Army post at West Point was called in to help. At his suggestion rockets containing a glass can of white and red lead were shot at the points where observation marks were desired. The results were most satisfactory.

Some of the difficulties on the second survey are evidenced by the fact that it took six days to cross-section 1,000 ft., whereas during average progress on ordinary roadwork a cross-section party will survey a mile a day.

For 700 ft. along the face of the mountain no cross-sections could be taken by ordinary means. For this work a special diamond-shaped target, 1 ft. on a side and 1 ft. across, was made of heavy-gage galvanized iron. The target was divided vertically and horizontally into four parts, painted alternately red and white. Half-inch holes were cut near the acute points of the diamond, and through these holes the target was fastened to 300 or 400 ft. of braided cord and pulled over the ledges.

When at the point desired, the target was bisected by three transits located at known points and the vertical and horizontal angles read and the time noted simultaneously by the three transitmen. Two of the transits were located on the West Shore railroad tracks about 600 ft. apart; the third (for a check) was placed a part of the time on the face of the mountain and finally at the base of the mountain. The movement of the target was governed through the cord by two men, one

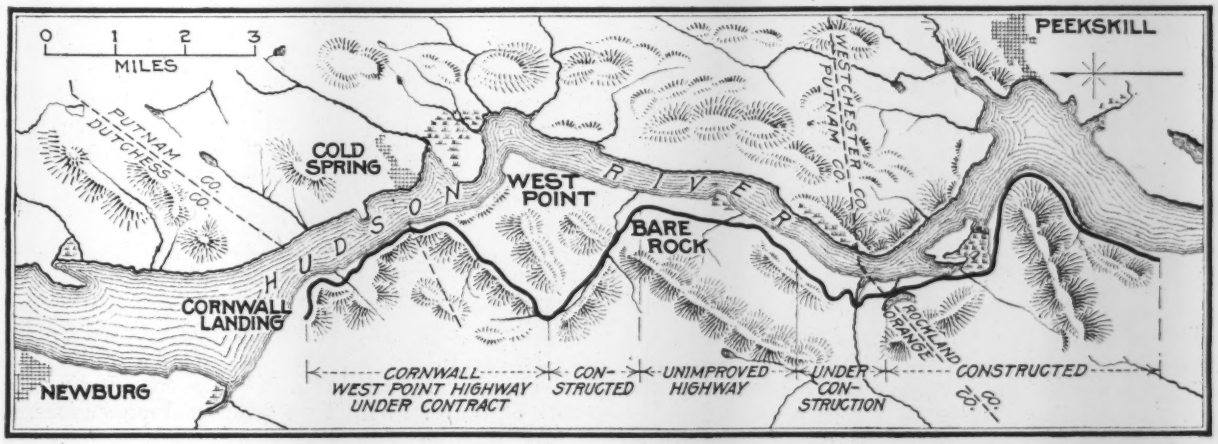


FIG. 6. MAP OF THE PROPOSED HIGHWAY ON WEST SIDE OF HUDSON RIVER

above and one below the ledge. It was found impracticable to take these cross-sections at right angles to the base line, for there were only a few niches in the ledges from which the ends of the target cord could be operated.

Description of Proposed Construction Work

According to present plans, the road starting at Bayview Ave. in Cornwall rises gradually along the side of the mountain on an average grade of about 5½% until an elevation of nearly 390 ft. above the river is reached at Sta. 38. This is one of the controlling points, for at a point a few feet east of the center line there is a sheer drop of nearly the whole 390 ft. This is just above the point where the toe of Storm King reaches out into the river.

The grade then descends at about 7% or less to an elevation of about 200 ft. at Sta. 68, which is in a cove. From here the grade is undulating around Crow's Nest Mountain, but on the average descends gradually to an elevation of about 150 ft. at the reservation line of West Point. From here the line skirts along the base of Crow's Nest Mountain, partly on new location and partly along the existing Washington Valley road to its intersection with State Highway No. 5,328A about 2 mi. west of Highland Falls.

The usable width of roadway will be 24 ft., with 16 ft. of bituminous macadam penetration method. In general the roadbed will be constructed in sidehill cross-sections partly cut into solid rock and partly carried by retaining walls, the design being worked out to the greatest economic advantage. The maximum grade is 7%, and the minimum radius of curvature is 200 ft. The greatest depth of rock cut measured from ditch line to the top on the inside of the cross-section is 130 ft.; the highest retaining wall is 38 ft.; the greatest depth of fill is 75 ft. The estimated total amount of excavation is 119,000 cu.yd., of which about 50,000 cu.yd. is ledge rock—much of the remainder is large boulders. There is 25,500 cu.yd. of stone masonry of different classes, about three-fifths of which will be laid dry. A greater part of this is for retaining walls.

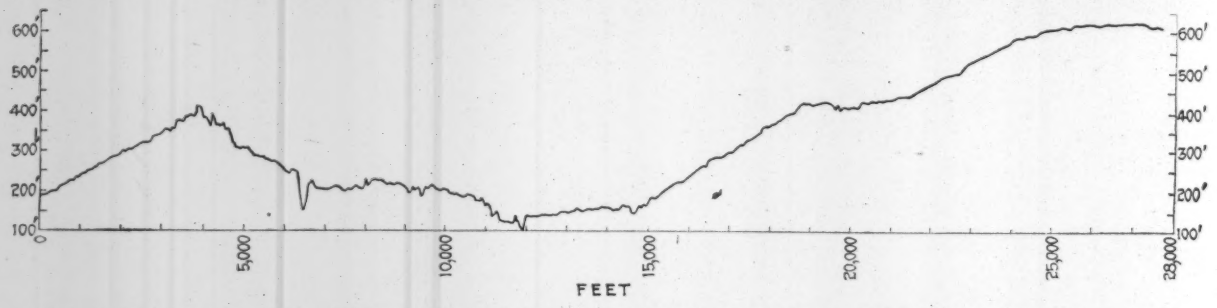


FIG. 7. PLAN AND PROFILE OF PROPOSED STORM KING ROAD, NEW YORK STATE HIGHWAY DEPARTMENT

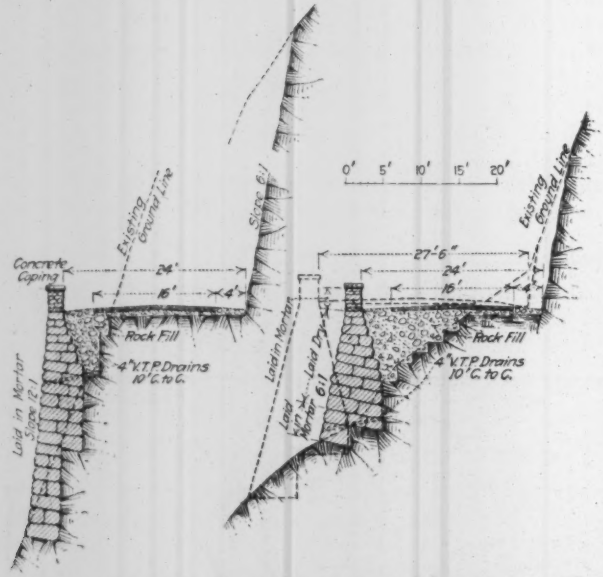


FIG. 8. TYPICAL CROSS-SECTIONS ON THE PROPOSED STORM KING ROAD

Unusual Provisions in Specifications

Specifications for this work have a few features uncommon in state-road construction. The classification of material varies from earth with small pieces of rock to earth with boulders up to a cubic yard in size, to all boulders and ledge rock. In order that there may be but few disputes after the work is done and to get a more equitable classification and estimated price for the different grades of material, there are four classifications of combined earth and rock excavation designated, ledge rock (in the same instances) being divided into two classes—excavation for the prism and excavation for culverts, footings of retaining walls and steps under fills.

The specifications provide for furnishing, installing, maintaining and operating a traveling crane and standard-gage car of at least ten tons' capacity on the railroad tracks at the foot of the mountain during the full period of excavation and wall construction along the face of Storm King and a section of the face of Crow's Nest.

It is also provided that the contractor shall install compressed-air lines leading to the West Shore railroad tracks, of sufficient capacity to furnish 300 cu.ft. of free air at 60 lb. pressure per min. to facilitate the removal of boulders or rocks from the tracks. It is also specified that while work is in progress on the sections directly over the tracks the "safety and continuity of the operation of traffic on the West Shore R.R. shall be of the first importance and shall at all times be protected."

It is provided that the contractor shall arrange for a telephone connection to be installed from the watchmen's building to the nearest train dispatcher of the railway company in order that communication may be had at all times with the superintendent of the railway to advise him of blasting and other operations affecting the safety of trains. With these plans and precautions it is hoped that the Storm King Road may be completed and opened to the public within the next two years.

Reinforced-Concrete Viaduct at St. Louis, Mo.

By Charles W. Martin*

SYNOPSIS—This 1481-ft. viaduct has a deck slab and beams carried by cross-girders on two-column and four-column bents. A special feature is that it consists of two independent longitudinal sections, separated by a 1-in. open space in the deck and cross-girders. The presence of sewers made it necessary to avoid all footings along the center of the structure. The concreting at one end was done with long inclined chutes from an elevator tower. At the other end it was done by a cableway handling drop-bottom buckets discharging into a hopper with short chutes.

The 12th St. viaduct at St. Louis, Mo., is one of a number of long bridges carrying streets across the group of railway tracks in the Mill Creek valley. The new structure consists of 36 reinforced-concrete slab and girder spans and one 58-ft. plate-girder deck span. It replaces an old steel structure. The length is 1481 ft. between abutments, or 2225 ft. including approaches. The approach grades are 2.3 and 1.8 per cent. L. R. Bowen is Bridge Engineer. The writer was in charge of the design, and Winters Haydock was in charge of the construction. The Fruin-Colnon Contracting Co. had the contract, and Preston Richardson was the company's superintendent. The total cost, including pavement, light and trolley poles, etc., was about $385,000.

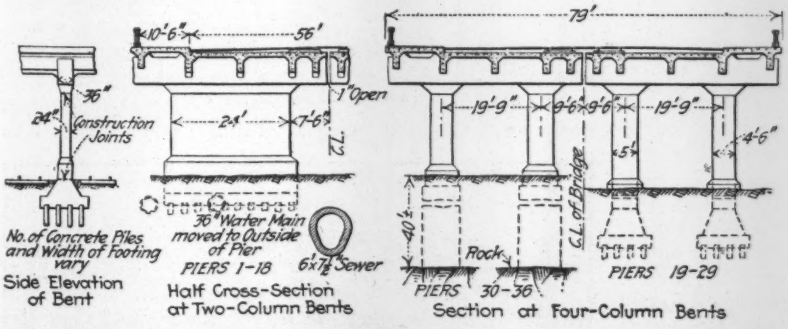


FIG. 1. TYPICAL SECTIONS OF THE 12TH ST. VIADUCT AT ST. LOUIS, MO.
Note the 1-in. open joint in cross-girders and deck. The viaduct consists of two parallel but independent sections

Varying Foundation Conditions

Tests indicated that the character of soil and depth to bedrock varied considerably and that at some points settlement might be expected from soil loadings exceeding about 1 ton per sq.ft. A large portion of the site near Gratiot St. was formerly occupied by a pond, and here no indications of rock were shown by borings 71 ft. below the filled-ground surface. The dead-weight loading of the soil-bearing piers of the old bridge was about 1¼ tons. Their settlement is unknown.

Four 30-ft. test piles of 10x10-in. timbers showed an average penetration of 4 in. under the last 25-ft. drop of the 1700-lb. hammer. From this a safe load of 9 tons per pile might be computed from the *Engineering News* formula. Test loads were applied to two of these piles. One settled ⅝ in. in 30 days under a load of 28 tons. The other was progressively loaded with 26 tons in a period of six days, and after having stood six days under full load it showed a settlement of ¼ in.

Most of the piers are built on reinforced-concrete piles 25 to 35 ft. long, but at the south end some of the piers were carried to rock. For the former, each pile was assumed to carry 15 tons of the dead-load. The gross average soil bearing was noted, however, and endeavor made to distribute the gross bearing area (as well as the piling) properly about the dead-loading centroid.

Design Adopted for the Viaduct

The design was governed largely by the sewers, a space in the center of the street being reserved in which sewers might remain or be built and upon which the bridge foundations could not encroach. This developed the idea of building the viaduct as two independent longitudinal sections, with piers on each side of the sewers, as shown in Fig. 1.

For the north half of the structure each bent consists of two piers, one for each section of the bridge. For the remainder each bent consists of four piers, as shown. The piers are 2 ft. thick, 24 ft. long in the two-pier bents and 4 or 5 ft. in the four-pier bents. All are reinforced longitudinally and laterally to resist the flexure and shear resulting from unbalanced deck loads and temperature distortions, as well as their direct axial load.

The reinforced-concrete deck spans vary from 28 to 54 ft. c. to c. of piers, and consist essentially of a slab and five stringers for each half-width of deck, as shown in Fig. 2. The roadway portion of the slab is 12 in. thick. Its top is flat and 12 in. below the street grade, allowing street-railway track construction with 7-in. rails and 4-in. steel ties of I-section. But it requires considerable filler concrete for the base of pavement and thereby augments the cost and dead-weight of structure considerably. At the curb the slab is stepped up to 3½ in. below the sidewalk grade, which allowed the independent construction of sidewalk blocks.

The stringers, slabs and cross-beams of piers were designed to be cast integrally for each half-width unit of bridge deck between consecutive expansion joints. At the girders the inside track stringer and the outside railing stringer overhang the pier about 5½ and 6½ ft. respectively. This cantilever arrangement compelled heavy construction for the pier girders. The outside stringers are of uniform depth for the sake of appearance. Those carrying the roadway are 28 to 56 in. deep beneath the slab, depending upon the load carried.

*Engineer, Bridge Division, City of St. Louis.

Stringers and girders are reinforced generally with two layers of bars, and at points of maximum shear in stringers about half of the bars are bent diagonally. Except for piers at expansion joints, such an amount of reinforcement is extended over and beyond the pier beams as to develop about the same figured compression unit in the bottom of the stringers as that in the slab near mid-

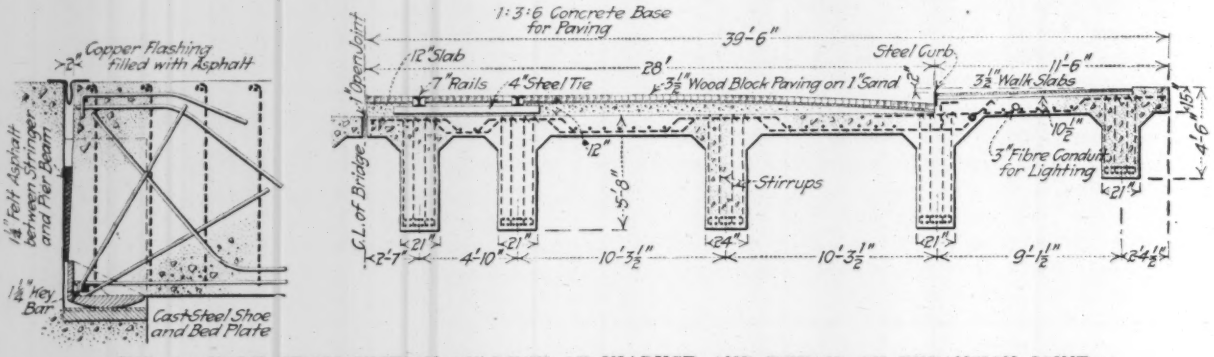


FIG. 2. HALF CROSS-SECTION OF DECK OF VIADUCT, AND DETAIL OF EXPANSION JOINT

dead- and live-load, flexure, etc.; shear in concrete, 120 lb. per sq.in.

The foundation piles were all precast and varied in length from 16 to 35 ft. The specifications called for a section equal in area to a 15-in. circle with 1¼-per cent. reinforcement. The type most generally used was 13½ in. square, with beveled corners, without taper, and reinforced with four ¾-in. deformed round bars hooped with No. 8 wire at 2-in. pitch. These were fabricated at the site by the contractor. Another type was the "corrugated" piling having a fluted or ribbed section. These were delivered on cars and were employed in pier foundations adjacent to the tracks. The heads of all piles were given extra reinforcement by cylindrical hoops of $\frac{3}{16}$-in. plate steel with riveted joints. These hoops were tried experimentally at first and proved to be a better preventive against shattering of pile heads than the usual extra wraps of hooping reinforcement. They obviated the necessity of an adjustable clamped driving head and thereby effected a considerable saving of time in driving.

The driving equipment consisted of a steam hammer in 45-ft. leads mounted (with boiler and hoist) upon a roller carriage. The weight of moving part in this hammer was 5000 lb. and its drop 42 in. A lighter hammer was tried and discarded. The number of piles driven was 1,980, a total of 51,300 lin.ft. The average performance was 330 lin.ft. and the best record 720 lin.ft. per 8-hr. day. In

span. This reinforcement is augmented by vertical stirrups, the stringer stirrups engaging the bent-up portions of the transverse bars of the slab. Expansion joints have a maximum spacing of 120 ft. The stringers of the free-end span are tenoned into the adjoining pier beam, and steel shoes and bedplates are provided, as shown in Fig. 2.

Other Features of the Structure

No reinforcement is extended across the 1-in. space that divides the structure longitudinally, but a 1-in. opening is left in the pier girders as well as in the deck slabs. This obviates the shear which might be generated in a full-width monolithic structure by unequal settlement of foundations on either side of the sewers. This opening was covered with an 8-in. plate of galvanized iron before the roadway concrete was laid.

The bridge deck provides a 56-ft. roadway and two 10-ft. sidewalks, the former being paved with wood blocks. Drainage is provided by cast-iron scuppers at the curbs, with cast-iron pipes in the piers and leading to the sewers. Combination trolley and lamp posts of limestone concrete are set along the curbs at intervals of 106 ft. Their surfaces are treated with acid. The lamp brackets and wire fastenings are of bronze. The 58-ft. span has ten steel girders, carrying an 8-in. concrete slab which conforms to the crown of the roadway. The girders are protected and masked by concrete so as to harmonize with the general structure.

The loadings employed in the design were as follows: A 56-ton street-railway cinder car; an 18-ton road roller; a uniform load of 125 lb. per sq.ft. of roadway; a uniform load of 100 lb. per sq.ft. of sidewalk. Wind pressure was not considered. The given loadings augmented by a nominal live-load impact allowance (varying 20 to 40 per cent. for different ratios of live- to dead-load effect) generate the following principal figured maximum unit stresses (bending effects were computed on the simple beam basis and unit stresses determined by the generally used straight-line theory): Tension in steel at points of low and high shearing stresses, 16,000 and 12,000 lb. per sq.in.; flexural compression in concrete, 650; axial compression in concrete, 400 for dead-load only and 700 for

FIG. 3. CONCRETING THE 12TH ST. VIADUCT AT ST. LOUIS, MO.

At the right are the cableway and drop-bottom bucket used for one end of the viaduct. At the left is the elevator tower of the spouting system used at the other end.

128 piles (6.5 per cent. of the total) less than 20 per cent, of the length was broken by the hammer, while 89 piles (4.5 per cent. of the total) had 20 per cent. or more of their length broken. For the latter, extra piles were driven. No marked difference in the driving resistance of the two types of piles was observed. The percentage of breakage in the fluted tapering type exceeded that for the other, but differences in methods of fabrication and curing probably account for this.

Concrete Mixing and Distributing Plants

Two independent concrete mixing and distributing plants were installed on opposite sides of the railway yard. Sand and gravel were delivered in cars. A stiff-leg derrick with clamshell bucket handled the material from cars to the stock pile and thence to the bins over the mixers.

These plants had different methods of distributing the concrete, shown in Fig. 3. At one plant (with a mixer of 18-cu.ft. capacity) there was an elevator tower 175 ft. high, with a system of suspended chutes. At the other plant (with a 1-yd. mixer) a cableway 700 ft. long with 60-ft. A-frame towers handled the concrete in 1-yd. bottom-dump buckets. The buckets were placed on trucks and pushed a short distance out from the mixer by hand; then picked up by the cableway and carried to a portable receiving hopper, from which the concrete was delivered to the forms by means of short chutes. Under fair working conditions the average output was 32 and 22 batches per hour for the tower and cableway plants respectively, so that their output of concrete was practically the same, say 22 cu.yd. per hr.

The plans indicated construction units, each of which was to be poured in one continuous run. Each pier footing constituted one unit, each pier shaft another, and each portion of the deck structure (together with the pier girders for one-half width of bridge between consecutive expansion joints) constituted a third. These deck units included one to three spans, and in some cases required as much as 350 cu.yd. of concrete. For these larger units work continued through two or more shifts of eight hours each.

The only feature of note in the formwork was, that for spans over the railway yard steel I-beams from the old structure were employed as the principal longitudinal elements. These were supported by framed bents adjacent to the piers and generally at points between tracks.

Small Sweeping Machines for Power and Hand Operation

The two sweeping machines shown herewith are designed for cleaning the floors of industrial plants, and also for driveways and yards. They are built by the Briggs-Chicago Co., 108 South La Salle St., Chicago.

The broom (of Japanese fiber) is carried by radial rods, so that it can adapt itself to an uneven surface, and it can be raised and held clear of the floor while the machine is being moved. A spring counterbalance provides for taking up the wear of the broom and for adjusting its pressure to suit the quantity and character of dirt to be handled.

The dirt is swept into a transverse pan 4 in. from the floor, and from this a conveyor (on the power machine) delivers it to a bin. In the hand machine the pan is raised by a lever to dump the material into the bin. In both cases the bin is of 2 cu.ft. capacity, and a hinged door in the housing provides for dumping dirt from waste cans, etc.

The power sweeper, shown in Fig. 1, has a 3-hp. Ellis gasoline engine which is equipped with a magneto and requires no batteries. It is 63x34½ in. over all, with a maximum height of 39 in., and weighs 700 lb. with its supply of gasoline, oil and water. At the normal engine speed of 600 r.p.m. the machine moves at 2 m.p.h. and will sweep 27,000 sq.ft. per hr.

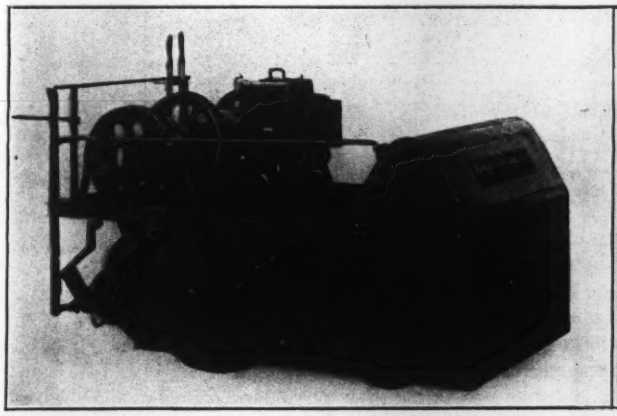


FIG. 1. POWER-OPERATED SWEEPER FOR FLOORS AND DRIVEWAYS

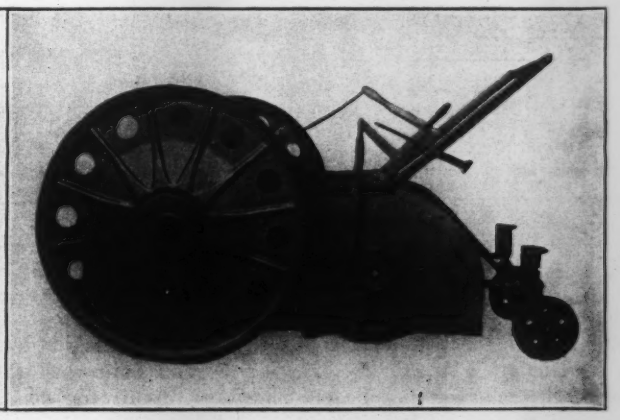


FIG. 2. HAND-OPERATED SWEEPER FOR FLOORS AND DRIVEWAYS

The power-operated sweeper is carried on three steel wheels having shock-absorbing duck tires. They are all within the 30-in. track of the broom, which can work to within 4 in. of a curb or wall. The broom is 18 in. diameter and 30 in. long. The engine is geared to the countershaft, from which the broom shaft and rear axle are driven by nickel-steel chains, but the machine can be moved by hand when desired.

The hand sweeper, shown in Fig. 2, is 40x56 in., and 34 in. high, and weighs 300 lb. The two 30-in. front wheels are of pressed steel, with rubber tires and ball bearings. The rear is supported by a pair of 7-in. caster wheels with rubber tires and having spiral springs on the casters. Cut gears are used, with a nickel-steel driving chain to the broom shaft. A pressure of 26 lb. on the handle bar is sufficient to propel the machine at 2 m.p.h., sweeping 20,000 sq.ft. per hr. at this rate. When the broom is raised clear of the floor a pressure of 15 lb. is sufficient to move the machine.

Rebuilding the Burlington's Platte River Bridge

By J. H. Merriam*

SYNOPSIS—Very long plate-girder bridge. Reconstruction involved the building of new piers and utilization of some of old plate-girder spans moved temporarily 14 ft. downstream to carry traffic during reconstruction. Variation in pier design. Steel caissons and concrete caissons used and special type of concrete piles in some piers.

The bridge for the Chicago, Burlington & Quincy R.R. across the Platte River near Ashland, Neb., was first built in 1886, as a pile trestle with sixty-two 20-ft. wooden stringers and six 105-ft. Howe truss spans. In 1898 the east channel, which was dry at this time, was filled and the bridge was shortened on the west end, making a total length of 1,643 ft. In 1901 it was rebuilt with 28 steel deck-girder spans on pile piers, protected by rock-filled cribs. The girders were of lengths varying from 50 to 60 ft., so as to use as far as possible the old piers which supported the Howe spans.

During the high water in the spring of 1912, an ice gorge formed at the bridge, causing the river bed to scour to such a depth at one point as to undermine one of the bridge seats, which was washed out. Fortunately the girders which it supported were not dropped into the stream, but were held over the opening by the track rails acting as suspension cables (Fig. 2). Temporary repairs were soon made but rebuilding was immediately decided upon. The new bridge is also of the plate-girder type, but has an entire new set of piers, the construction of which presented many novel problems.

A peculiar characteristic of the Platte River is that it invariably has adjoining one bank a main channel and against the other bank a secondary channel. These two channels are from one-quarter to one-half mile apart with stretches of extremely shallow water and sand bar between. The banks are low and very seldom overflow, except at times of ice gorges in the early spring. The fall is great and the current rapid and during high water the amount of silt carried is considerable. The sand is rather heavy, although not so heavy as to prevent its being readily scoured when the stream is confined.

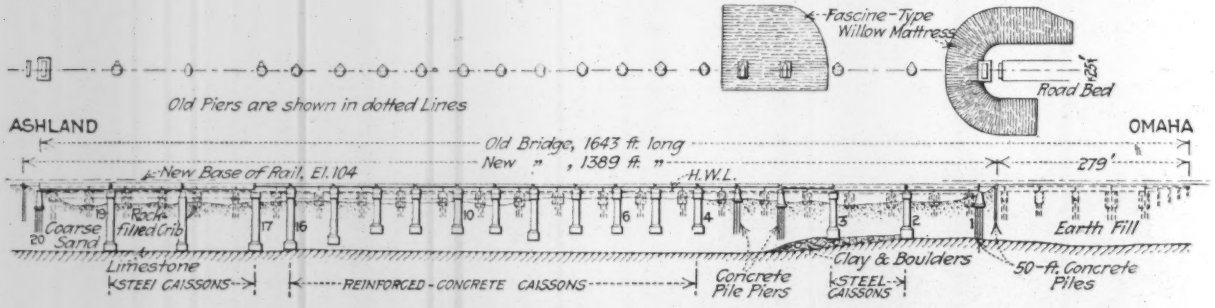


FIG. 1. THE CHICAGO, BURLINGTON & QUINCY BRIDGE ACROSS THE PLATTE AT ASHLAND, NEB.
Dotted lines show old bridge and solid lines new bridge

*Resident Engineer, Chicago, Burlington & Quincy R.R. Co., Bridge Department, Kansas City, Mo.

FIG. 2. THE OLD ASHLAND BRIDGE WITH THE WASHED-OUT PIER IN BACKGROUND

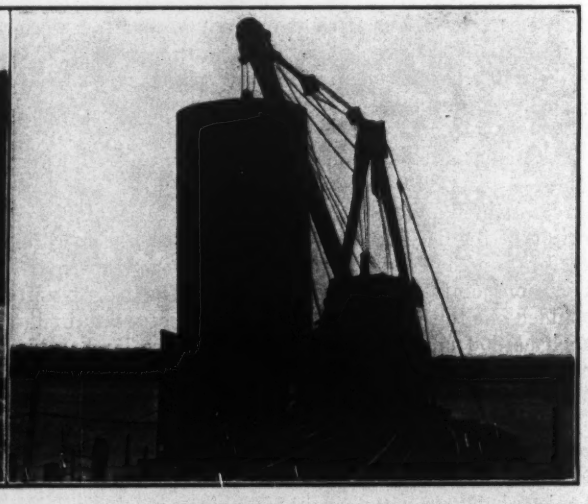


FIG. 3. PLACING 20-FT. SECTION OF STEEL CAISSON FOR ASHLAND BRIDGE PIER

For this reason it has been found the better practice to narrow the stream as much as prudence will permit, causing the channel to scour, providing at the time of the spring breakup wider and deeper openings for the passage of the very heavy ice which often forms, and reducing the width of the sand bars between the channels to the minimum. These characteristics determine the design of this bridge in large measure.

It will be noted from the elevation of the bridge in Fig. 1 that on either side very deep foundations are carried to bedrock or very light hardpan, while in the middle of the bridge, where the depth would in ordinary streams be the greatest, the foundations are carried only to a stratum of coarse sand or fine gravel which has demonstrated its bearing power and in which location the probability of scour is extremely remote.

The narrowing up of the original bridge by about 300 ft. was expected to result, and has resulted, in slightly increasing the depth of the channel in low water and it is anticipated that a further increase in channel depths will be secured during the next floods.

Type of Bridge

Before beginning construction it was necessary to move the entire bridge 14 ft. downstream to care for traffic while constructing the new piers on the old line. The new work comprised 20 new piers so located as to miss the old bridge seats. Deep piers were placed for spanning the two channels, using one 75-ft. and two 105-ft. spans over the east or secondary channel and one 103-ft. and two 100-ft. spans over the west or main channel, with one 50-, five 55- and seven 60-ft. girder spans from the old bridge for the remainder or middle portion of the bridge. The girders were spaced 7 ft. c. to c. and carry the single track on a solid concrete deck with ordinary ballast tie and rail construction. The deck slabs were premolded and placed on the bridge in 5-ft. sections. Each slab measures 14 ft. wide by 10½ in. thick, is reinforced throughout and provided with two 1¾-in. drain holes in each section. Joints between slab sections are calked with oakum, tar and cement. Expansion joints at the ends of each span are covered with apron plates.

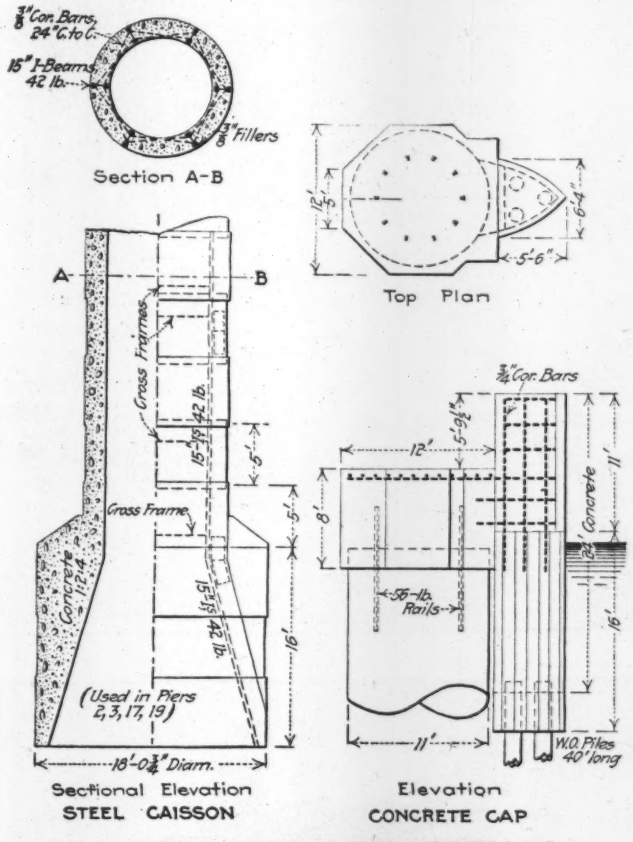


FIG. 4. STEEL CAISSONS AS USED ON PIERS 2, 3, 17 AND 19

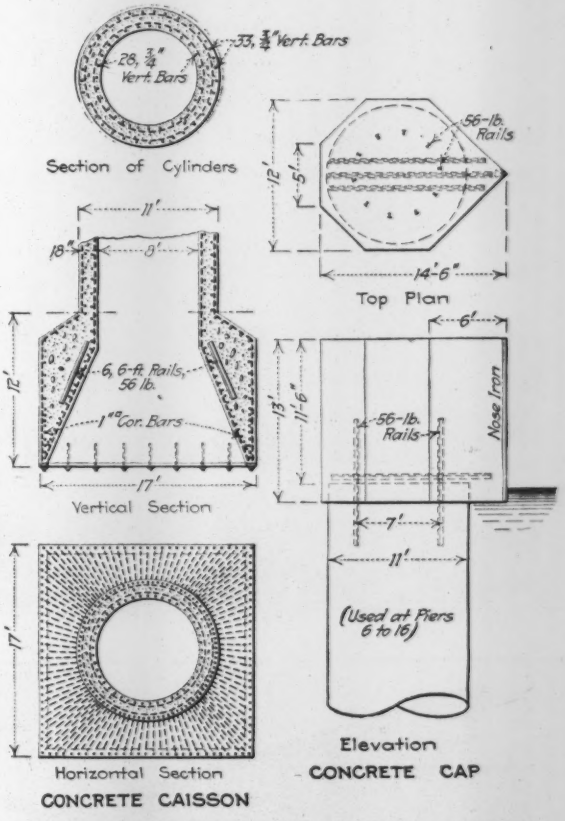


FIG. 5. CONCRETE CAISSONS AS USED ON PIERS 6 TO 16

The new bridge is 4½ ft. higher than the old in order to give a full open waterway under the long girders. Raising the bridge necessitated elevating the approaches, which are now on a 0.4 per cent. grade for a distance one-quarter mile east and one-half mile west from the bridge. Fascine-type willow mattresses, as shown in the plan in Fig. 1, were used to protect the new fill at the east end of the bridge and to maintain a channel under the long spans at this point.

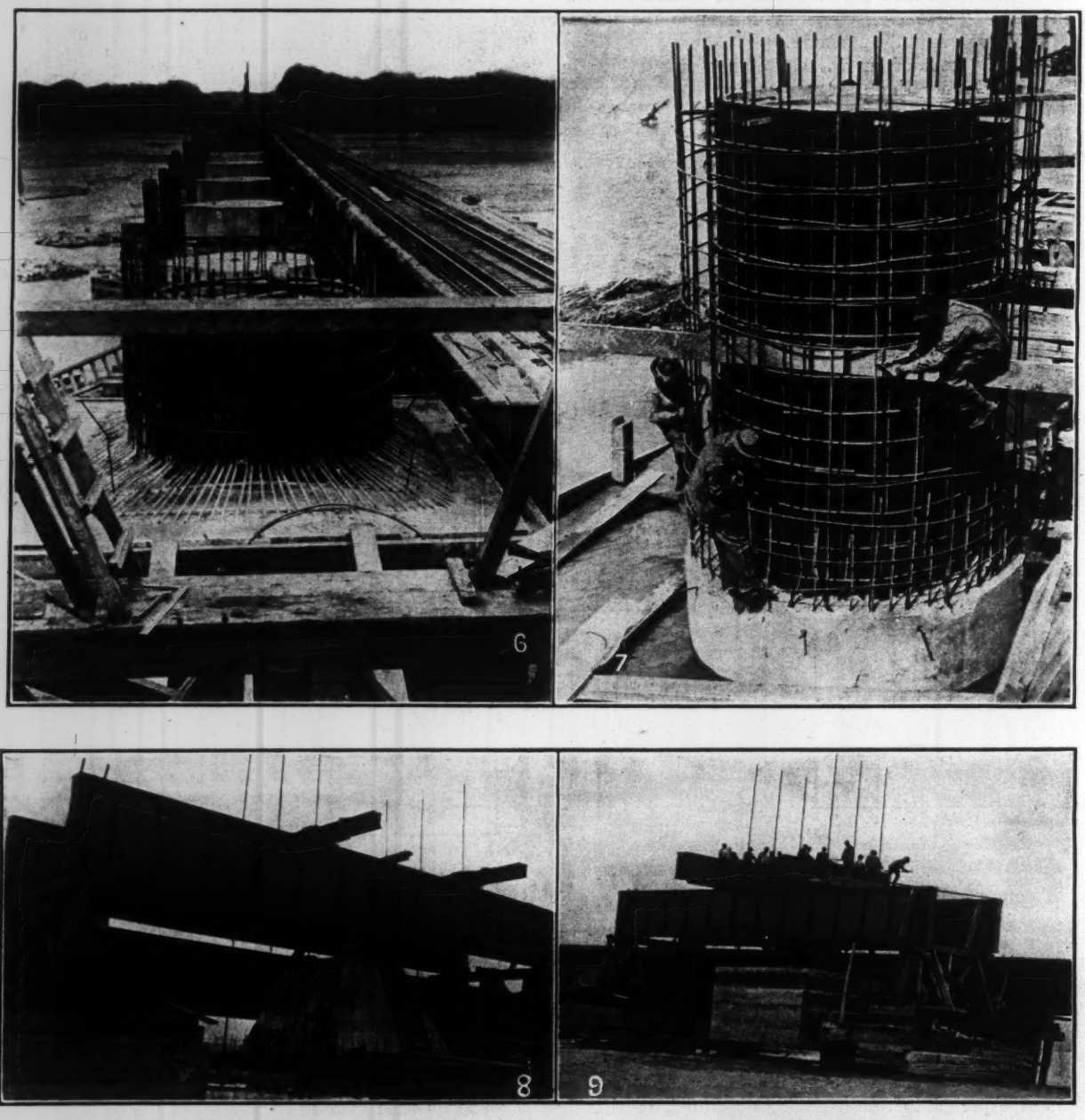
This is the first bridge of its kind to be constructed in a permanent manner on the lower Platte River and in order to obtain data for future work various types and methods were used in constructing these piers.

Caissons for piers 2, 3, 17 and 19 were of steel construction of the design shown in Fig. 4. They were lined with 18 in. of concrete, leaving an 8-ft. well, through which a dredge bucket was operated from a track derrick

on a temporary bridge or from a stiff-leg derrick and hoist when that rig could be erected.

The bottom section of the caisson was assembled on the bank and was carried out and set in place by a locomotive crane. Twenty-foot sections were added to this as sinking progressed in the manner shown in the view in Fig. 3.

The caissons were sunk by open dredging through the 8-ft. well in the center and by shooting one ½-lb. stick 40% dynamite in the excavated pit after each 5 to 8 cu.yd. of sand was removed. By using dynamite, it was possible to keep the sand from rushing in at the bottom; it seemed, also, to help materially to sink the caisson. The perpendicular position of the caisson was maintained by regulating the excavation in the bottom and by means of shores against the outside. At times it was necessary to employ divers to remove rock, boulders and logs from under the cutting edges. Excavation was usually from 3 to 4 ft. below the cutting edge when the caisson started to move. In order to found the cutting edge on bedrock, the last 3 or 4 ft. of depth was made by lowering the water on the inside, thereby reducing the pressure and allowing the sand to be forced in under the cutting edge and permitting the caisson to settle by its own weight to within a few inches of bedrock. Sand around the cutting edges which could not be removed was then thoroughly grouted



FIGS. 6 TO 9. THE BURLINGTON'S ASHLAND BRIDGE UNDER CONSTRUCTION

Fig. 6—Looking across bridge with concrete caisson in foreground. Fig. 7—Inner form and reinforcement in place for 13-ft. section of concrete caisson. Fig. 8—Pier 11, showing suspended platform with forms partly erected. Fig. 9—Pier 11, lowering bottom of sections of caissons by screws

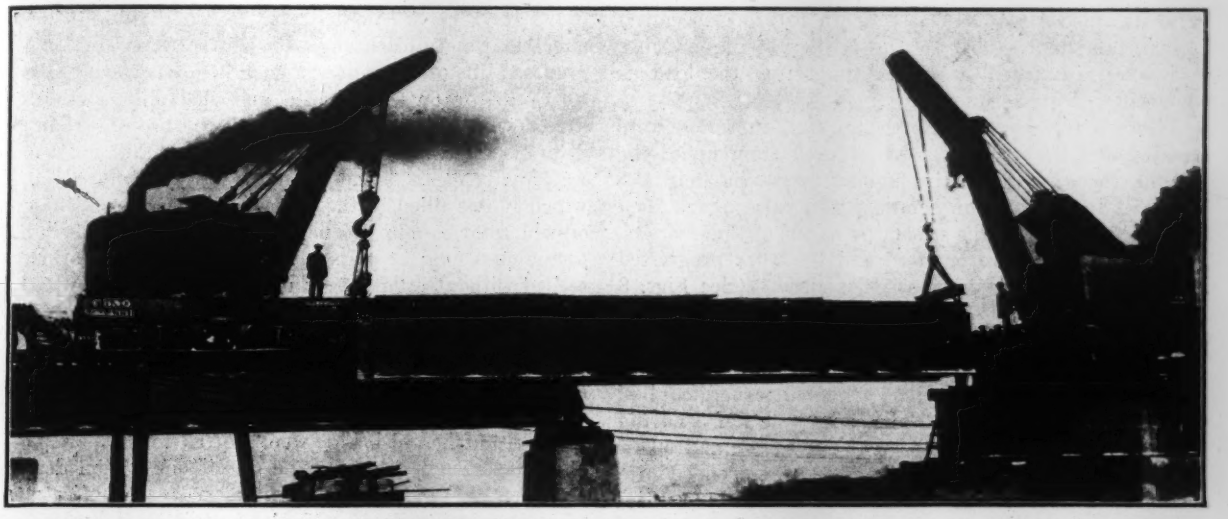


FIG. 10. MOVING GIRDER SPANS ON CHICAGO, BURLINGTON & QUINCY BRIDGE ACROSS THE PLATTE RIVER FROM OLD TO NEW PIERS

with cement, applied through a 2-in. steam jet. The bottom was then sealed with a 5-ft. course of 1:2:4 mix placed through the water in a ¾-yd. drop-bottom bucket. After this had set for 12 hr. a second course was placed, making a total of from 10 to 14 ft. of sealing material. This was allowed to set for 24 hr. when the caisson was unwatered and filling with concrete was carried on in the dry. A mix of 1:3:6 was used to complete this dry filling of the core of the piers.

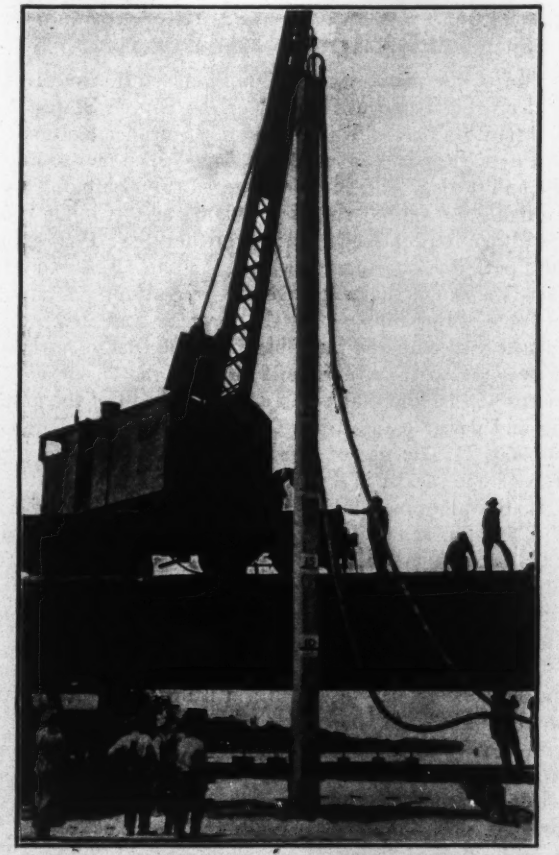


FIG. 11. SINKING SPECIAL CONCRETE PILE ON ASHLAND BRIDGE

The piers were completed to proper height by placing reinforced-concrete caps with ice-breakers on the upstream side. With a track derrick using a 1-yd. Browning orange-peel bucket a maximum of 6 ft. was sunk in 10 hr., which includes 25 per cent. loss of time on account of traffic. Where a stiff-leg derrick and hoisting engine were used, a maximum of 10 ft. in 10 hr. was made, with no loss of time on account of traffic.

Reinforced-Concrete Caissons

Caissons for piers 6 and 16 inclusive were made of reinforced concrete with a 17-ft. square base converging to an 11-ft. cylinder 12 ft. above the cutting edge. All forms for molding this caisson were of wood and so constructed that they could be removed and used again. Details of these caissons are given in Fig. 5.

They were sunk and finished in the same manner as the steel caissons previously described. Pier 16, only, was founded on bedrock; pier 15 was founded on a stratum of cemented gravel about 10 ft. above bedrock, and the remainder of the series were sunk to a depth of about 50 ft. below low water and founded on coarse sand or gravel, which is below any possible line of scour. Owing to the shallow water it was possible to build diversion dams of brush and rock upstream from where a new pier was to be started, thereby obtaining still water in which a wooden box coffer-dam, measuring 24 ft. square and from 4 to 6 ft. deep, was placed and filled with sand, on which forms for the bases of the caissons were erected. After sinking the caisson to a depth of from 10 to 12 ft. this coffer-dam was removed and used again at the next pier.

Piers 6 to 16, except No. 11, were started in this manner. At pier 11 the water was 12 ft. deep, so it was decided to erect base forms and concrete the lower section of the caisson on a suspended platform. This was accomplished by spanning the location of the new pier with two 60-ft. girders placed at 13-ft. centers on pony bents built on the old bridge seats. Crosswise on these girders were placed two pairs of smaller girders from which a platform was suspended just above the surface of the water, by means of six 2-in. lowering screws 24 ft. long. The form for the base was erected on this platform and con-

crete was placed to a sufficient depth to properly embed and anchor six eye-bars to which six 2½-in. lowering screws were attached to aid in supporting the load as concreting progressed.

After the forms were removed, the six 2-in. screws were released and the platform was removed from under the caisson, throwing the entire weight (approximately 175 tons) on the six 2½-in. screws. The caisson was then lowered to the bed of the stream by means of these screws, after which they were removed and the work progressed as on the other piers previously described. (See Figs. 8 and 9.)

Concrete-Pile Footings

Piers 1, 4 and 5 are built on Bignell concrete piles, pier 1 and pier 4 having 36 piles each and pier 5 having 21 piles, sunk to depths respectively of 57, 62 and 50 ft. below water. The tops of these piles are embedded in a concrete cap, on which forms were erected and piers finished in the usual manner.

The concrete pile referred to is of a new type (see *Engineering News* of Oct. 8, 1914) and these are the first of their kind to be used. They are 16 in. sq., 50 ft. long and weigh 12,300 lb. each and are made of reinforced concrete. Each pile has a 4-in. pipe through its length which is reduced to 2 in. at the bottom. This is tapped by ¾-, ½- and ⅜-in. pipes fitted with elbows turned up on the outer edge of the piling and are spaced from the bottom or point of pile, 1½, 2, 4, 8 and 12 ft., thus furnishing five openings on each side of the pile. Also a 2-in. pipe extends through the 4-in. pipe and fits into the reducer at the point, the 2-in. pipe being fitted with a brass nozzle that reduces the opening to 1⅛ in.

Water is applied at the top of the piling, through two 3½-in. hose, at a pressure of 100 to 150 lb. per sq.in. in the 4-in. pipe and 250 to 300 lb. in the 2-in. pipe (see Fig. 11). The high pressure is used at the point of the pile, through the inner pipe, to loosen and displace sand in front of the pile in sinking. The lower pressure in the 4-in. pipe supplies the side openings with enough pressure to keep the sand loose and to give it an upward movement to overcome the side friction. The pile sinks by its own weight.

Abutment Design

The east abutment consists of a prismatic pier resting on 50-ft. Bignell concrete piles, 12 of which were sunk to a depth of 35 ft. below low water, supporting the east end of a 25-ft. slab, weighing 80 tons. The other end of the slab is supported by a small cone block on brackets attached to ends of girders. Two of these slabs are used, one at each end of the bridge. They were manufactured at the Chicago, Burlington & Quincy R.R. concrete plant at Havelock, Neb., each slab being made in two parts measuring 3 ft. deep, 7 ft. wide by 25 ft. long to facilitate handling and placing.

The west abutment was built the same as the east one, except that it is supported by twelve 30-ft. by 14x14-in. solid concrete piles.

Pier 20 is similar to pier 1, but the foundation piles are white oak instead of concrete.

Pier 18, which was the last pier to be completed, presented many difficult problems, owing to the nature of obstructions in the river bed, through which it was sunk. A coffer-dam, 19 ft. square, was planned, but before driving, a Davis calyx core drill was used to drill holes 18 in. apart to penetrate through the obstructions and on the proposed line of coffer-dam. Each hole was blasted with 5 to 15 lb. of 60% dynamite and sheetpiling was then driven with but little trouble. By the use of dynamite, divers, dredge buckets and sand pumps, this coffer-dam was finally cleaned and the obstructions removed, after which it was filled to the surface of the water with sand on which forms were erected for a concrete caisson similar to the ones used at piers 6 to 16. Thereafter work proceeded without interruption until the pier was completed.

Steel Erection

Three new spans were placed at each end of the new bridge, and track laid and completed as far as possible. Traffic was then diverted for 34 hr. while the girder spans from the old bridge were shifted to the new piers. This was accomplished by the use of one 75-ton derrick and one 100-ton derrick, which picked each entire span with ties and rails in place, raised it 7 ft., moved it endwise 25 ft. and swung it to place on the new piers, 14 ft. out. These three movements required 20 to 45 min. for each span. Thirteen spans were moved in this manner, during the worst storm of the winter. High wind and heavy snow greatly retarded this work. The actual time for changing the spans was 10½ hr. (Fig. 10).

The construction was all carried out by the forces of the Chicago, Burlington & Quincy R.R., under the direction of C. H. Cartlidge, Bridge Engineer; C. R. Fickes, Assistant Engineer. The writer was Resident Engineer.

Okanogan Pumping Plant Is Irrigators' Insurance

Two power plants and one pumping station have been built on the Okanogan Project of the United States Reclamation Service in Washington as insurance against the contingency of lack of water. About 10,099 acres of private and project land here are watered from Salmon Creek, requiring an annual runoff of 30,000 acre-ft. The creek has always maintained a runoff greater than this, except for a period of shortage between 1908 and 1914. In 1911 there was an actual shortage for irrigation. A supplementary water-supply for the project was determined upon to be secured by pumping from the Okanogan River. It is expected that there will be more seasons when the pumps will be idle than when used. The cost of the pumping and power plants when fully completed will be about $61,000. If the use of this plant should save the apple crop of one season it would pay for itself.

In the pumping station are two units, each consisting of a 200-hp. induction motor direct-connected to a two-stage centrifugal pump of 6 sec.-ft. capacity. The two pumps are connected by 12-in. pipes to a Y in a 30-in. line, 225 ft. long, and a wood-stave pipe 4,400 ft. long. The water is raised 177.5 ft. above low water.

To supply the pumping station with current two generating stations have been built at drops in the irrigation-canal system. At one plant there is a drop of 105 ft. with a normal water flow of 27 sec.-ft., developing approximately 250 hp. At this plant the machinery comprises a Francis turbine connected to a 187-kv.-a. generator. At the second power station there is a drop of 56 ft. and a flow of 55 sec.-ft., developing about 300 hp. The equipment is the same as noted in the first plant.

Cost of Drainage Pumping in Southern Louisiana

By C. W. Okey*

SYNOPSIS—The costs of operation of drainage pumping plants in southern Louisiana are given with full data on the equipment of each station and on the kind and costs of fuel and labor. To facilitate the comparison of plants of different capacities and heads the unit used is the cost of lifting 1 acre-ft. through 1 ft. height.

The data set forth in this article have been obtained during the course of investigations carried on by the writer in southern Louisiana since June, 1909, under the auspices of Drainage Investigations, Office of Experiment Stations, United States Department of Agriculture. The work has consisted of a general study of the problems involved in the reclamation of the wet prairie lands of southern Louisiana, particular attention having been paid to the subject of run-off, including its amount as determined by continuous records kept, and the cost of its removal by pumping.

The equipment of each pumping plant, as well as the kinds and costs of fuel and labor, are shown in Table 1. Except for the last three districts enumerated, the descriptions apply to conditions obtaining in 1912, but any important changes since made have been noted. Table 2 shows the costs of operation, including those of fuel, labor and repairs; it also shows the conditions under which the plants were operated. For the purpose of comparing the costs for the various plants, a unit has been chosen which eliminates any effect of difference in the amounts of water pumped and the lifts of the pumps; this unit is the cost of lifting 1 acre-ft. of water 1 ft. The performance of each plant is expressed in the equivalent number of acre-feet lifted 1 ft. It should be noted that the table shows average effective lift rather than the actual, the former being the difference between the elevation of the water surface from which the water is being pumped and that of the water surface where the pumps discharge. The cost of pumping is likewise expressed as the cost of lifting 1 acre-ft. one effective foot.

Notes on Cost Tables

Prior to 1913 figures for the costs of repairs for the Willswood and the Little Woods plants were not available, and it has therefore been assumed that such costs were 15% of the combined cost of labor and fuel. This is an average figure for the plants for which the actual costs of repairs are known. An unusually large amount for repairs and incidentals is recorded against the Little Woods plant for the year 1913, as this item covers a thorough overhauling of the plant; strictly speaking, this extra expense should be charged to the whole succeeding period of operation instead of to the one year.

Prior to September, 1913, the design of the suction and discharge piping at the Poydras plant was such that the resulting loss of head amounted to about 2.5 ft.; with a better design of piping this loss was reduced to about 0.5 ft., with consequent increase of efficiency.

The area drained on Unit No. 1 at Gueydan was changed in July, 1915, from 6,500 to 7,500 acres by the addition of about 1,000 acres of raw land; consequently the run-off from the district as a whole was a very large part of the total rainfall. Fuel oil during most of the year cost $1.50 per bbl. (42 gal.) by the time it was barged to the plant. By means of an oil meter a careful record was kept of the amount of fuel required for each boiler to bring the steam pressure up to the point necessary to operate the plant; this was found to be 55 gal. for each time. Labor was estimated at $0.50 per fire-up, so that the average cost for each boiler of the plant was $2.50.

The Only Electric Plant

On the Little Woods tract the area was decreased about 10% at the beginning of 1913, and on the first of May it was further reduced to 2,400 acres. The area drained by the plant continued to be 2,400 acres to the close of the year, except during the month of October, when 6,858 acres was drained. The service of the plant was very intermittent as it was desired to obtain the deepest drainage possible. Consequently the reservoir channels were kept as nearly empty as practicable. From the first of the year 1914 and until the middle of May, this district was drained by two plants. The old plant drained an area of 2,400 acres and the new plant drained the remaining 4,543 acres. After the date mentioned the entire area was drained by the new plant. This is the only electrically driven plant on which records were taken. The motors are of the fixed-speed type, so that the speed of the pump remains constant whatever the lift. This arrangement gives the greatest capacity when the lift is low, at which time the greatest capacity is necessary. While the capacity decreases very materially with the increased lift, the water to be pumped also greatly decreases.

The cost figures given in the table must be compared with due regard to conditions of operation and to the average lifts of the pumps. A comparison of the costs of lifting 1 acre-ft. of water 1 ft. on the Little Woods tract for the years 1910 and 1911 will show the increase of cost that results from very intermittent operation. The increase in this case is due largely to the relatively large charge for labor during 1910. However, it will be noted that the fuel charge is greater for 1910 than 1911.

The effect of a change in the average lift of a pump is shown in the figures for nearly all the plants, but especially in those for Unit No. 1 Poydras, No. 2 Des Allemands, and No. 3 Raceland. On some of the other plants where an increase in lift has not caused a decrease in unit cost the cost of fuel has increased.

As all of the Corliss-engine plants are on large districts, thus causing a relatively large reduction in the labor charge per unit of water lifted out of the district, it will be of more value to make the comparison on cost of fuel alone. This will largely eliminate the effect of the size of the district. With the exception of the year

*Senior Drainage Engineer, United States Department of Agriculture, Washington, D. C.

TABLE 1. EQUIPMENT DETAILS AND UNIT PRICES OF OPERATING EXPENSES; SOUTHERN LOUISIANA DRAINAGE DUMPING PLANTS

Plant and Location	Pumps: Number and Kind	Pumps: Capacity: Size	Pumps: Capacity: Total Sec. Ft.	Pumps: Capacity: Depth, In. per 24 Hr.	Excess Lift (Actual Exceeds Effective), Ft.	Engines: Power Transmission	Engines: Number and Kind	Engines: Size	Efficiency of Engine, Pump and Piping at Aver. Lift, Per Cent.	Boilers: Number and Kind	Boilers: Size	Fuel: Kind	Fuel: Unit Cost	Labor: Grade	Labor: Wages	Labor: General Plant Condition	Notes
Smithport, Lockport, La.	1 square wood-case centrifugal	32x12-in. impeller	...	....	2.0	Rope drive	1 slide-valve noncondens	11x12-in.	Not tested low	1 return-tubular	100 hp.	Coal and oil (a)	$4.00 per ton	Unskilled	$1.50 per day of operation	Poor	Large plant, on account of leaking levees
	1 do.	24x8-in. impeller	34	1.1	.....	do.	1 do.	12x10-in.					$1.15 per 24 gal.	do.		do.	(a) Fuel changed from coal to oil in September, 1912
Willswood, Waggaman, La.	1 square wood-case centrifugal	42x16-in. impeller	...	....	5.0	Rope and bevel gear	1 slide valve noncondens	16x24-in.	12	2 water-tube	150 hp. each	Oil	$0.80 to $1.70 (b) per bbl.	Good	$3.00 per day of operation	Good	(b) In 1912, cost of fuel oil rose from $0.80 to $1.70 per bbl.
	1 rotary	7 ft. wide	...	....	5.0	Rope drive	do.	16x24-in.	32	1 return-tubular	80 hp.	do.		Unskilled assistant	$1.50	do.	
	1 horizontal cast-iron	36-in. discharge	152	1.4	2.0	Direct-connected	1 double slide valve vertical	14x14-in.				do.					
Unit No. 1, Poydras, La.	1 horizontal cast-iron centrifugal	36-in. discharge	...	....	2.5 (c) 0.6 (d)	Direct-connected	1 slide valve noncondens	14x18-in.	28 to 45 (e)	2 return-tubular	80 hp.	Oil and coal	$0.90 per bbl.	Engineer good	$1.80 per day of operation	New	(c) March, 1912, to August, 1913 (d) Since August, 1913
	1 do.	24-in. discharge	92	0.88	.....	do.	do.	10x14-in.	32 to 55 (e)				$3.25 per ton	Fireman fair	$1.00 per day		(e) Increased efficiency due to changes in pumps and piping during 1913
Drainage District No. 1, Des Allemands, La.	2 horizontal cast-iron centrifugal	24-in. discharge	75	0.95	0.3	Direct-connected	2 slide valve vertical	12x12-in.	55	2 return-tubular marine	60 hp. each	Oil	$1.15 per bbl.	Good	$1.50 per day of operation	Good	
Drainage District No. 3, Raceland, La.	2 horizontal cast-iron centrifugal	30-in. discharge	120	1.25	0.75	Direct-connected	2 slide valve vertical	14x16-in.	35	2 return-tubular	100 hp.	Oil	$1.15 per bbl.	Unskilled	$1.50 per day	Fair	
Unit No. 1, Gueydan, La.	2 horizontal cast-iron centrifugal	54-in. discharge	385	1.4	0.3	Direct-connected	2 Corliss non-condens	16x36-in.	55	2 return-tubular	150 hp.	Oil	$0.80 to $1.25 per bbl.	Expert engineer, good fireman	$100 per mo. in 1912; $3 per day $1.50	New	About January, 1913, the engineer was changed from monthly wage to rate per day when operating the plant
Little Woods (Old Plant)	1 horizontal cast-iron centrifugal	48-in. discharge	100	0.32	2.0	Direct-connected	1 Corliss noncondens	16x36-in.	2? (f)	1 water-tube	150 hp.	Coal	$2.75 per ton	Expert engineer, good	$90 per mo.	Good	(f) Test in 1909; change in pump and piping in 1911 probably much increased the efficiency
Little Woods (New Plant), Little Woods, La.	2 do.	48-in. discharge	380	1.30	0.6	Herringbone gear	Electric motors	215 hp. each	50				(g)	fireman	$1.50 per day		(g) 1c. per kw.-hr. plus $230 per month fixed charge
Jefferson Drainage District No. 3, Barataria, La.	2 horizontal cast-iron centrifugal	48-in. discharge	300	1.44	0.4	Direct-connected	2 Corliss noncondens	16x36-in.	55	2 return-tubular	150 hp. each	Oil	$1.15 per bbl.	Expert	$55 per mo.	New	

TABLE 2. COSTS AND CONDITIONS OF OPERATION; SOUTHERN LOUISIANA DRAINAGE PUMPING PLANTS

1	2	3	4	5	6	7	Cost: 8	9	10	11	Per Acre-Ft.: 12	13	14	15	16	17	18	19	20							
Plant	Area, Acres	Year	Rainfall, In.	Run-off, In.	Aver. Effective Lift, Ft.	(a) Acre-Ft.-Ft.	Fuel	Labor	Repairs and Incidentals	Total	Per Acre	Total	Fuel	No. Days Plant Operated	No. Times Boilers Fired Up	No. of 24-Hr. Days per Year at Full Capacity	Hr. at Full Capacity per Fire-Up	Longest Period of Continuous Operation, Days	No. of times plant was run continuously more than following No. of days: 1	2	3	4	6	10	Notes
Smithport	647	1911(b)	57.52	33.35	5.2	9,400	$667	$351	$150	$1,168	$1.80	$0.125	$0.071	115	103	33	7.7	2	8	1	0	0	0	0	(a) Acre-feet of water lifted 1 ft.
	647	1912(c)	52.23	40.80	5.5	12,190	795	342	170	1,307	2.02	.108	.066	108	77	49.5	12.9	4.3	12	5	3	1	0	0	
	647	1913	60.01	37.77	5.8	11,840	780	215	35	1,030	1.60	.087	.066	86	68	34.2	12.0	3.8	10	3	1	0	0	0	(b) March, 1911, to February, 1912, inclusive
	647	1914*																							
	2,400	1909(d)	43.32	16.33	5.0	16,330	936	600	230	1,766	.73	.108	.057	45	39	20.0	7.3	2.5	3	1	0	0	0	0	
	2,400	1910	43.08	11.58	5.0	11,580	616	500	180	1,296	.54	.112	.053	45	41	8.4	4.9	0.75	0	0	0	0	0	0	(c) March to December, inclusive
	2,600	1911	52.32	23.42	5.0	52,290	1,267	753	300	2,320	.89	.092	.050	69	61	19.2	7.6	3.50	5	1	1	0	0	0	
	2,600	1912(e)	48.22	34.42	5.0	37,290	2,676	765	514	3,955	1.52	.106	.072	81	63	35.0	8.9	4	10	4	1	1	0	0	(d) June to December, inclusive
	2,600	1913	51.52	27.08	5.0	30,180	2,412	848	457	3,717	1.43	.123	.080	68	60	17.5	7	2	6	2	0	0	0	0	
	2,600	1914	42.25	15.05	5.0	16,320	1,210	426	464	2,100	.81	.129	.074	41	39	9.7	6	2.2	2	1	0	0	0	0	(e) January to May and October to December, inclusive
Unit No. 1, Poydras	2,500	1912(f)	70.52	28.28	2.1	12,420	1,098	640	244	1,982	.79	.160	.088	114	77	41.8	10.5	14.0	12	9	7	4	1	1	
	2,500	1913	63.45	38.82	4.5	36,580	2,314	1,261	347	3,922	1.57	.107	.063	155	131	46.5	8.5	6.5	4	3	2	1	1	0	
	2,500	1914	47.70	16.46	5.2	18,320	743	764	408	1,915	.77	.105	.041	73	71	17.8	6.0	2.0	2	1	0	0	0	0	(f) March 9 to December 31
Drain. Dist. No. 1, Des Allemands	1,880	1913	47.37	31.00	5.1	24,910	1,526	564	116	2,206	1.18	.089	.061	210	198	40.0	4.8	5.0	5	4	2	1	0	0	
	1,880	1914	45.70	25.83	5.3	21,340	871	517	65	1,453	.77	.068	.041	178	159	35.3	5.3	4.8	8	5	2	1	0	0	(g) March 17 to December 31
Drain. Dist. No. 3, Raceland	2,250	1913	52.04	20.84	2.9	11,270	930	306	331	1,567	.70	.140	.082	90	90	15.5	4.1	0.75	0	0	0	0	0	0	
	2,250	1914	58.95	23.40	3.5	14,870	1,140	233	194	1,567	.70	.106	.077	78	71	18.7	6.3	1.60	3	0	0	0	0	0	(h) Area increased to 7,500 acres in July
Unit No. 1, Gueydan	(h)6,500	1912(g)	41.71	37.26	3.6	75,200	2,640	1,225	795	4,660	.72	.062	.035	196	185	40.0	4.1	4	6	3	2	1	0	0	
	(h)7,500	1913	77.61	65.86	5.0	194,000	6,013	1,321	631	7,965	1.06	.041	.031	No record	169	58.0	8.2	No record	No record						(j) October to December, inclusive
	7,500	1914	59.18	34.49	5.1	109,350	No record			5,175	.69	.047	.....		166	162	28.00	4.1	5.2	7	1	1	1	0	0	
Little Woods	7,500	1908(j)	3.26	3.88	1.5	3,730	137	381	77	595	.08	.160	.037	23	23	.....	9.2	0.5	0	0	0	0	0	0	(k) Average for year
	7,500	1909	65.40	29.17	3.0	55,610	1,168	1,629	420	3,217	.43	.058	.021	233	193	82.00	10.2	8.5	11	8	5	3	1	0	(m) January 1 to May 2; old plant
	7,500	1910	51.25	7.94	4.9	24,380	652	1,303	295	2,250	.30	.092	.027	134	134	31.00	5.5	0.5	0	0	0	0	0	0	
	7,500	1911	53.85	18.36	4.7	53,620	1,060	1,505	385	2,950	.39	.055	.020	163	142	52.00	8.8	7.0	8	6	4	3	2	0	(n) Average for year; new plant
	7,500	1912	63.50	42.39	5.1	136,650	3,100	2,121	670	5,911	.79	.043	.022	294	204	145.00	17.1	23.0	7	7	5	4	4	2	
	(k)4,260	1913	46.04	24.28	5.6	62,390	1,375	1,565	1,000	3,940	.92	.063	.022	182	180	60.00	8.0	2.0	2	1	0	0	0	0	(p) Electric power
	(m)2,400	1914	17.40	13.13	5.6	14,770	No record			2,120	.88	.143	.....	59	58	39.00	5.8	1.2	1	0	0	0	0	0	(q) March 12 to December 31
	(n)6,040	1914	50.60	20.46	4.5	46,760	3,932(p)	No record		5,650	.....	.121	(p) .084	308	308	16.00	1.2	0.6	0	0	0	0	0	0	
Jefferson Drainage Dist. No. 3	(q)5,000	1913	49.60	28.42	5.1	59,970	2,565	692	480	3,737	.75	.062	.043	121	119	29.00	4.9	2.5	1	1	0	0	0	0	*No record
	5,000	1914	48.16	26.45	4.7	51,630	2,274	595	261	3,130	.63	.061	.044	104	97	18.3	4.5	2.9	5	1	0	0	0	0	

1914 the cost in fuel of lifting 1 acre-ft. of water 1 ft. was much less for the Corliss-engine plants than for those which use slide-valve engines. The cost is especially low for Little Woods, because of the low price of fuel. The longer run per fire-up for this plant gives it further advantage in some of the years. The plants in District No. 1 at Gueydan and Jefferson Drainage District No. 3 are similar in equipment except that the pumps in the former are larger. It will be noted that the cost of fuel used per unit of water lifted is less for the Gueydan district than for the other. The lifts and the lengths of run per fire-up are about the same, while the unit cost of fuel is lower for the Jefferson plant. The difference may be due to the faulty firing in the latter, but it probably is caused by the fact that while the engine at Gueydan is of about the correct size for the 54-in. pump, the same-sized engine connected to a 48-in. pump in the Jefferson Drainage District plant is rather larger than necessary and is underloaded.

Slide-Valve Engine Plants

For the slide-valve plants, a comparison of the costs of fuel for lifting 1 acre-ft. of water 1 ft. will show that the effect of the highest average lift and the longest run per fire-up for the year 1913, which occurred in the case of the Smithport plant, has served to make the latter compare very favorably with the other slide-valve plants—although it is by far the poorest plant of the five. On the other hand, the effect of the lowest lift and the shortest run per fire-up for the year 1913, which occurred in the Raceland plant, has served to make it the most expensive plant per acre-foot of water lifted one foot—although the Raceland pump is the equal of any of the other slide-valve plants. It will be noted that the unit fuel costs for the four slide-valve plants from which cost figures were available showed, for 1914 as compared to 1913, some decrease in all cases and a decided decrease in two. In the case of Willswood this decrease was caused by a cheaper price on the fuel oil burned, and in the other three by increased lift and greater care in operation. Tests of the plants had showed that they were being operated at an overload, and during 1914 the engine speeds were considerably reduced. At Poydras changes in the piping to the pumps and in the impellers also aided in achieving more economical operation. It will be noted that the cost figures for the plants at Poydras and Des Allemands are about the same as those for the Corliss-engine plant at Jefferson.

The only plant operated by electricity shows a considerably higher cost for power for lifting 1 acre-ft. of water 1 ft. than do most of the steam plants. However, it will be noted that the amount pumped was small and the service was very intermittent. The plant was started about twice as often as any of the others and the average length of run at full capacity for each time it was started was less than one-fourth as long as in the case of most of the others. Of course the loss due to starting so often was much less relatively than with a steam plant.

The conditions of operation as set forth in Table 2 may also need some explanation. Column 17 gives the total time in each year that the plant was operated at full capacity, expressed in terms of full 24-hr. days. Where the period given in column 3 is less than a year, the length of time that the plant actually operated during the period is so corrected as to give the rate per year. For example, if the plant operated 10 days in a period of 6 months, the number that would appear under column 17 would be 20. The figures in column 18 are obtained by dividing the actual number of hours during which the plant operated in the given period by the number of times the boilers were fired. Where the period given in column 3 is a full year, the figures in column 18 may be obtained by multiplying those in column 17 by 24 and dividing by the numbers given in column 16. If the period is less than a year the figures in column 17 should first be corrected by column 3 and notes.

Conditions on some of the districts during the various years are such that the amounts and costs of operation are not typical and could not safely be used for drawing general conclusions. Thus in the case of Smithport the amount pumped and the number of 24-hr. days per year that the plant operated are both increased by seepage through the levees; this is particularly true during 1912. The length of run per fire-up is also increased by the same influence and under normal seepage conditions would probably be about one-third less. The Willswood plant has been in operation about 19 years; the number of days per year it operates (an average of 17) and the average length of run per fire-up (7 hr.) are about what would be expected for lands that have been reclaimed for a number of years. The Poydras, Gueydan, Des Allemands and Jefferson plants are all on newly reclaimed lands. Up to the beginning of 1914 no very large percentages of the lands in the respective districts were in cultivation, and the character of the service and the number of days per year that the plants operated are comparable only to the same items in plants on newly reclaimed land. The conditions of operation at the Little Woods plant were far from being typical up to the beginning of 1914, as its capacity was relatively small and the water was allowed to come to it very slowly from the portions of the district not in cultivation. During 1914, since the addition to the plant capacity, the district could be drained in about the usual time.

Some Typical Results Shown

In the year 1914 the results from the Poydras, Des Allemands, Jefferson and Raceland plants may be said to be typical. In addition, the results from the last-named district for 1913 are also typical. An average of the records for 1914 from each of the districts and including records for 1913 from the Raceland district shows:

Rainfall, in.	50.85
Run-off, in.	22.59
Days per year on which operation occurred	104.00
Number of times per year boilers were fired	97.00
Number of 24-hr. days per year at full capacity	21.10
Average number of hours run at full capacity, per fire-up	5.20

While the rainfall given is slightly below normal for this section of the state, it is believed that the percentage of rainfall appearing in run-off in the average given will not be far from normal. With the mean annual rainfall of 56 in. and the same percentage, the amount to be pumped would be 25 in. per yr. Therefore a plant with a 24-hr. capacity of 1.25 in. in depth of water over the area drained would have to operate an average of about 20 days of 24 hr. each, per year. Where such factors as leaky levees enter in, the amount of pumping would of course be greatly increased. Where the capacity of the plant is much less than specified, the amount pumped would be somewhat less, owing to evaporation.

Flooding and Recovery of the Astoria Tunnel--II

By Harold Carpenter*

SYNOPSIS—Tunnel pumped out, while some leakage still continued, by combined air lift and pump work. Erratic pressure-gage readings in the bulkheaded heading. The jammed bulkhead door and the open drain-pipe. Leak finally grouted through a 22-ft. drill hole from the Astoria bulkhead. Excavating the grout filling the heading—a volume twice that of the cement used. Final bench work and lining with cast-steel rings.

For pumping out the tunnel after the leak had been controlled by filling the entire heading with grout, the following preparations were made: One 1,000-gal. and one 800-gal. pump were mounted on a steel pontoon in the Astoria shaft. An 18-in. ventilating pipe and three 8-in. pump discharge lines which were already in the shaft were converted into air lifts by installing small air lines in them. The three elevators were removed so that the elevator framework could be used to mount additional pumps.

On Nov. 24, 1913, at 7:50 a.m., the 18-in. and two 8-in. air lifts were turned on. They lowered the level of the water 160 ft. by 9 p.m., removing 1,140,000 gal., an average of 1,440 gal. per min. The 1,000-gal. pump was then cut in. As the work progressed two pumps were used from the pontoon assisted by the air lifts and three 600-gal. pumps mounted on the elevator framing. The discharges of the pumps were connected to discharge lines already in place in the shaft for the original pumping plant by means of 8-in. hose, and steam connections were made in the same manner. The bays of the elevator framing were 25 ft. apart, and after the water was pumped down low enough so that the three pumps could be shifted, the pumps on the pontoon were used to hold the level of the water while the shift was being made. Conversely, the pumps on the elevator framing were used to hold the water while the connections of the pontoon pumps were being changed.

When pumping began, a gage on a pipe in bulkhead *B* (see sketch, Fig. 4, p. 675, last week's issue) read 92 lb., the exact figured pressure for that point. In the early pumping this gage dropped slower than the head in the shaft by 19 lb. A 5-in. pipe in bulkhead *B* was opened in order to assist the pumping by using the Bronx pumps, when the gage began to rise rapidly until it was 24 lb. higher than the shaft pressure, indicating that the flow was opening up connection with the river pressure. The pipe was closed, and the gage jumped to 58 lb. higher than the shaft pressure.

At 10 a.m., Nov. 25, the 18-in. air lift was shut down, having operated until the water was 200 ft. below the top of shaft, which was an unusual performance, as the air pipe only extended down a distance of 256 ft. About 2 p.m. the amount of water coming into the shaft appeared suddenly to increase, as if some stoppage had given way in the heading.

On Nov. 26, as the gage at bulkhead B had been continually dropping and had apparently lost its connection with the river pressure, the 5-in. pipe was again opened and the Bronx pumps operated, when the gage instantly dropped 13 lb. more.

As the Astoria heading was about 20 ft. higher than the Astoria portal, it was feared that a vacuum would form in the heading and cause a rush of water that would swamp the pumps. In looking for a means of forcing air into the heading a 2-in. pipe in bulkhead *B* was cleared; and instantly the gage, which was on another pipe on the opposite half of the bulkhead, dropped another 15 lb. to the shaft pressure and then 3 lb. more.

The gage fluctuated back and forth—first higher, then lower than the shaft pressure—in a very extraordinary manner. On Nov. 28 when it should have read zero it read 7 lb. and continued to do so to the end of the pumping. When the water reached an elevation about 14 ft. above the bottom of tunnel at the portal, the submerged pumps were cut in. Approximately 24,000,000 gal. of water was pumped.

Vacuum in Heading Holds Back Water

The attempt to force air into the heading from the Bronx end had failed. At 3:30 p.m., Nov. 28, it became evident that a vacuum was forming in the heading, as the water level had reached a point at that time which it should not have reached until midnight. It was calculated that 1,000,000 gal. was held back in the tunnel above the elevation of the water in the shaft.

An attempt to force air through an air line which was known to extend well into the tunnel finally succeeded, most fortunately, when the air pressure reached 165 lb., evidently blowing out some stoppage, as the air then went in at 35 lb. Air was then kept constantly on this line, and lowering of the water proceeded.

The last of the 8-in. air lifts was shut down at 11:30 a.m., Nov. 29, having worked continuously from 7:50 a.m., Nov. 24. The water was 234 ft. below the top of shaft, and the end of the air pipe had only 13 ft. submergence, an unusual performance for an air lift.

Entering the Tunnel for Inspection

At 5 a.m., Nov. 30, when the water reached a point 4 ft. above the crown of the arch at the portal, the air which had been forced into the heading began to break into the atmosphere, bubbling up through the water violently. The pumps were slowed down and the escape of the air lowered the water to the level of the portal. The full 100 lb. of air was then blown into the tunnel to ventilate it. At 7:30 p.m. the water reached a level of 4.5 ft. below the crown of arch at the portal, which meant that it was 13.5 ft. deep at the portal, diminishing to nothing at a point 2,600 ft. from the portal, or 900 ft. from bulkhead *A*. As it was most important to know the actual conditions in the heading, the

*Resident Engineer, Astoria Tunnel; Astoria Light, Heat and Power Co., New York City.

Engineers' Agent and Resident Engineer manned two rowboats and with safety electric lamps proceeded to the heading, the pumps holding the level of the water. Heavy timber jams were encountered and had to be cleared away as the boats proceeded. On reaching the face of bulkhead *A*, it was found that, while the safety door had been dropped as reported, it had jammed on something (afterward found to be a small piece of wood) and had not closed tight. This, together with the 8-in. drain pipe, was responsible for flooding the Astoria end of the tunnel. Also, when the heading was pumped full of cement grout, the partly open door allowed the grout to run out into the main tunnel for a distance of 500 ft. A dam that had been built to hold drainage water while concreting the invert, stopped the flow of the grout along the tunnel invert so that the latter gradually formed a bank against the door and finally sealed it off. This bank was from 12 ft. deep at the bulkhead to 1 ft. deep 500 ft. distant, and varied from fairly well-set cement at the doorway to white mud farther away, through which one had to wade.

The inspection showed that the only leaks into the tunnel were through a 3-in. cable pipe (see Fig. 9) and through one of the 8-in. drain pipes (whose valve had not been closed, as already noted), amounting in all to less than 1,000 gal. per min., with practically no pressure. Through the open drain pipe and an 18-in. pipe that led away its discharge to a point 1,000 ft. distant there had run into the tunnel about 500 cu.yd. of green sand, coal and refuse. This was entirely separate from the grout. The party reached the shaft again at 9 p.m. The unwatering of the tunnel was completed at 4 p.m., Dec. 1.

On Dec. 3 a trench was excavated in the grout bank to get access to, and control of, the valves on the bulkhead drains, from one or more of which water was flowing. Two of the pipes were found closed, but the third pipe was open and running a part stream. As soon as the valve was moved, water broke out in large volume. The valve evidently had choked with rubbish from the leak and, fortunately, had remained so during the unwatering of the tunnel and the subsequent period. It was most fortunate that the partial stoppage of this pipe did not give way while the inspection party was in the heading on the night of Nov. 30. The condition of this pipe accounted for some of the peculiar and sudden changes in pressures and conditions which had occurred during pumping.

The large flow caused by disturbing the valve reduced the pressure at the gage in bulkhead *B* from 32 to 17 lb., showing direct connection between the two and showing also that the source of the leak had not been completely shut off from the river. The valve was then regulated so as not to disturb the pressure between bulkheads.

In the meantime work had been started on an additional safety bulkhead reinforced with rails, 60 ft. back of bulkhead *A*, for additional protection of shaft and tunnel. The pumping plant at the Astoria shaft was increased to a capacity of 9,800 gal. per min., which was greater than the boiler capacity.

Final Grouting of the Leak

On Dec. 16 drilling of test holes and core borings into the side of the tunnel just outside bulkhead *A* was begun, in an attempt to locate the water-bearing seam and grout it to shut off the remaining connection with the river. The test holes were calculated to be only about 20 ft. from the seam and were carefully directed to intersect the line of the seam as plotted from the data

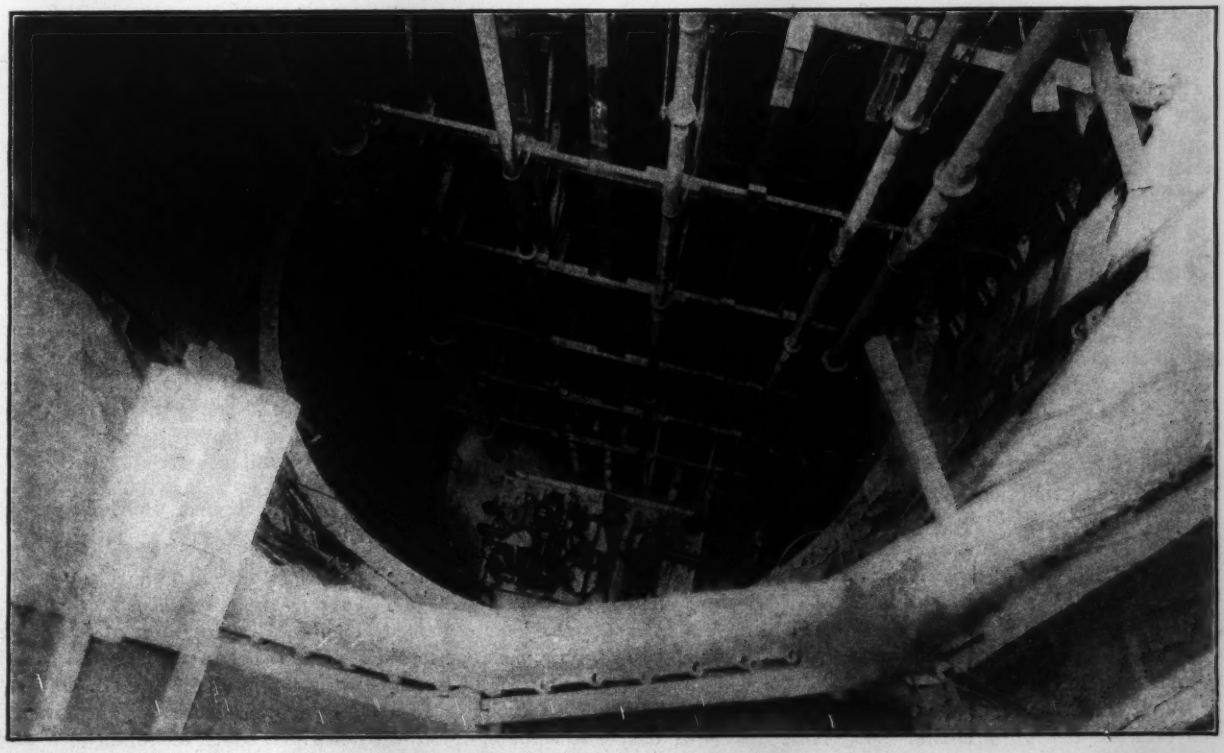


FIG. 8. PUMPING OUT THE ASTORIA SHAFT, NOV. 24, 1913
Three air lifts in operation. Pumps on pontoon started a few hours later

FIG. 9. BULKHEAD AND GROUT BANK

Dec. 1, 1913. Showing grout bank which piled up against bulkhead door and finally stopped flow of water. Door had jammed on stick of wood, allowing water and grout to come through. Drain pipes are 3 to 4 ft. below door

available. The core borings were aimed to reach the seam at a point further in, nearer the original water hole. About 17 test holes and 10 diamond-drill core borings were drilled into the side of the tunnel and through the bulkhead *A* into the bench near the water hole.

One of these test holes, about 22 ft. long, on Dec. 17 struck decomposed rock (green sand) and water under the full pressure. It was grouted with 3,351 bags of cement under high pressure. Conditions were noted by means of pressure gages on bulkheads *A* and *B*, and by permitting pipes in bulkhead *B* to remain open. Grout appeared at once at bulkhead *B*, showing that the grout hole was connected to the original leak and that the latter was still open into the heading. This grouting was effective in stopping the flow.

While the grout was being given time to set, a thorough exploration of the bench and heading was made through bulkhead *A* by means of core borings, but no further water was encountered. This work was completed on Jan. 4, 1914, three months after the flood of Oct. 5.

Excavation of Grout from Heading

On Jan. 5 excavation of the grout from the heading between bulkheads *A* and *B* was begun (see Fig. 7, p. 677, last week's issue). The bulkhead door was found jammed open by a stick of wood 4 in. by 1 in. The door was opened on Jan. 7. A drift through the grout was completed up to the door in bulkhead *B* and the door opened on Jan. 13. The entire space between bulkhead *A* and bulkhead *C* was filled with grout except for some sand at the bottom; and the space between bulkhead *C* and bulkhead *B* was filled with grout to within 2 ft. of the top. This grout was highly stratified, some of it being set fairly hard, while next to it would be a layer of grout of the consistency of chalk.

A most peculiar fact was that, while the total amount of cement used in grouting from both ends was only about 600 cu.yd., nevertheless 1,200 cu.yd. of solid grout was removed from the tunnel and the space between bulkheads, not to mention the quantity that evidently must have remained in the seams and stopped the flow of water.

This more than doubling in volume of the grout can only be explained on the assumption that cement in the form of grout does not maintain constant volume. The fact that the discharge of sand and débris from the leak was mostly carried 1,000 ft. away and kept entirely distinct and separate from the white material, and, further. that this discharge had no white material in it whatever, makes it improbable, in the writer's opinion, that the increased volume was due to a mixture of grout and dolomitic sand from the leak. The discharged material was all very dark in color, and the sand from seams had always been dark except in a very few instances when material similar to beach sand had come in.

Most of the grout was poorly set and was in fact largely removed with picks. The diamond bit was found embedded in a block of grout that in turn was buried under 4 ft. of green sand and débris, being completely separated from the main mass of grout. The removal of the grout was completed on Jan. 14.

The top of the remaining bench between bulkheads *A* and *C* was trimmed off 2 ft. below springing line and a concrete arch invert built over the entire top of the bench, bonded underneath the concrete arch lining. The purpose was to explore with test holes the entire bench and grout any water seams through the invert arch before excavating. This invert was completed Jan. 20.

Bench Excavation Resumed Under Protection of Temporary Invert

On Jan. 22 the excavation of the 23.5 ft. of bench between bulkheads *C* and *B* was resumed very slowly, preceded by many test holes. The drilling of test holes systematically located was also begun through the invert on top of the bench between bulkheads *A* and *C*. On Jan. 27 water and green sand under pressure was encountered in some of these test holes just south of bulkhead *C* and grouted. On Jan. 31 very bad ground was exposed in the west side of the excavation between bulkheads *C* and

B, and at 3 a.m., Feb. 1, water broke out and flooded this section to springing line but developed no pressure. The section was lined with thick concrete walls, draining the water through pipes that were later grouted.

After the top of the bench between bulkheads *A* and *C* had been thoroughly explored with test holes drilled through the temporary invert, three test-hole pits were excavated across the bench and concreted, in order to allow still deeper exploration with test holes from these pits. In all, 284 test holes were drilled in the bench between bulkheads *A* and *C*.

On Feb. 16 excavation of the bench in the danger section just north of the Astoria bulkhead *A* was resumed, and advanced very carefully and with as little blasting as possible. As soon as each section of about 20 ft. was excavated, it was immediately lined with concrete before proceeding farther. The rock of this entire section was very much disintegrated, wet and full of seams.

Seam That Caused the Flood Laid Bare

On March 2-4, 1914, the second section, 17 to 46 ft. north of bulkhead *A*, was excavated. The rock was very bad. The excavation on its westerly side exposed what was undoubtedly the seam that caused the flood of Oct. 5, 1913.

This seam was from a few inches to 2½ ft. wide, as exposed. Portions of it were filled with refuse exactly like what came in during the flood, other portions were filled with grout, and still others with a combination of grout and refuse. The west side of the excavation was very bad disintegrated rock and seams of sand and grout. At one point there was a synclinal fold which changed the stratification from 25° with the horizontal abruptly to the vertical. One seam was grouted solid 6 in. thick. Several flows of water occurred, during one of which a seam was exposed for an area of 7x9 ft. (apparently a vertical section) and a water hole washed out 3½ ft. square. They were stopped by grouting through pipes and drill holes without unusual difficulties.

Excavation of the bench was completed on March 19, except for a small portion underneath bulkhead *A*, which could not be removed until June 16, 1914.

Cast-Steel Lining in Trouble Zone

It was decided to line 400 ft. of the tunnel through the trouble zone with cast-steel rings. Work was begun on this Apr. 17, 1914, and completed Nov. 29. During the placing of steel rings the tunnel and shafts at each end were always kept protected by safety bulkheads, three additional ones being constructed for this purpose. No further flows of water occurred, however. The removal of these bulkheads was completed Nov. 24, 1914.

In the meantime the lining of the rest of the tunnel with concrete was going forward. The concrete lining (including the concrete inner lining of the steel rings) was completed Jan. 10, 1915. The first piece of the 72-in. gas mains was lowered and placed in position Jan. 23, 1915. The work of installing the two mains is now nearing completion.

Quantities of Materials; Accidents

Eight safety bulkheads across the full-section tunnel and three across the heading were constructed to protect the tunnel from floods. In addition 35 temporary bulkheads had been built over portions of the heading or bench to confine water for grouting.

The construction of the tunnel involved about 100,000 cu.yd. of excavation, including surface and miscellaneous, of which about 90,000 cu.yd. was rock. Of this 39,500 cu.yd. rock was crushed on the job, and largely used in the concrete lining. Concrete placed amounted to 36,000 cu.yd., of which 32,000 was for permanent lining. The lining required 200,000 bags of cement; foundations, bulkheads, etc., 27,000 bags; for grouting 118,000 bags was used; making a total of 345,000 bags.

Test-hole drilling to explore for water included 3,318 holes of a total length of 43,300 ft.; they were grouted

FIG. 10. TEMPORARY CONCRETE INVERT ON TOP OF BENCH BETWEEN BULKHEADS A AND C Jan. 22, 1914. Constructed for protection in exploration of bench for water before removal

with 26,700 bags of cement. Further, 17 diamond-drill holes totaling 1,500 ft. were drilled, consuming 1,800 bags of cement in grouting them. For grouting shaft and tunnel lining, both concrete and metal, 63,000 bags of cement was used. Grouting behind bulkheads consumed 26,500 bags.

For blasting, 195,000 lb. of dynamite was used, with a record of only one accident from its use, resulting in only one death. The coal consumption was 62,000 long tons (up to Jan. 1, 1915). About 226,000,000 cu.ft. of water was pumped out of the tunnel, representing a consumption of approximately 628,000 hp.-hr. for pumping alone.

Notwithstanding the extremely hazardous nature of the work, extending over a period of 4½ yr., only six fatal accidents occurred, of which only one was due to dynamite and none to the dangerous conditions due to floods. This is perhaps a unique record for construction work of a similar nature, and is due to the unusual precautions taken for safety.

Inception of Project and Organization

The credit for the inception and carrying out of the project of providing for the ultimate removal at a future date of all the gas plants from Manhattan to a great central plant on Long Island (the Astoria plant) and the conveying of gas from this plant to Manhattan and Bronx by means of tunnels is entirely due to William H. Bradley, Chief Engineer of the Consolidated Gas Co., who originally conceived this plan nearly 30 years ago and, with the completion of the tunnel described in this article, now sees it finally certain of being carried to completion.

The entire work of engineering, organization and construction was carried on by the Consolidated Gas Co., of New York, through its subsidiary, the Astoria Light, Heat and Power Co., under the direction of William H. Bradley, Chief Engineer; W. Cullen Morris, Engineer of Construction; and Harold Carpenter, Resident Engineer; by Jacobs & Davies Inc., as Tunnel Engineers, who engineered the Pennsylvania Railroad Hudson River tunnels, the Hudson & Manhattan Ry. tunnels and other similar work in other parts of the world. Vivian Messiter, Engineer's Agent, was in charge of the Astoria tunnel construction work for Jacobs & Davies.

Chicago Breakwater Extensions

The establishment of a lake-front harbor at Chicago and the construction of the municipal pier, described in *Engineering News* of July 29, 1915, necessitate the provision of breakwaters to form an inclosed area and afford protection for vessels approaching or moored at the pier. The present breakwater is a stone-filled crib structure about 5,500 ft. long, extending northwest from its outer end toward the shore. The new work provides for closing the gap at the north end and extending the breakwater from the south end. The work is under the direction of Lieut.-Col. W. V. Judson, United States Corps of Engineers, Chicago. G. A. M. Liljencranz is United States Assistant Engineer at Chicago.

Timber-Crib Breakwater

The extension to close the gap between the present breakwater and the shore will consist of 2,250 ft. of stone-filled crib running due west, leaving a 400-ft. opening for ships between the old and the new work and extending to within 540 ft. of the shore. At present only the outer 1,500 ft. of this is to be built. The depth of water is from 19 to 24 ft., and the lake bottom is of clay covered by from 12 to 24 in. of sand.

The construction is shown clearly in Fig. 1. The cribs will be 30 ft. wide and 100 ft. long. The five cribs at each end will be 18½ ft. high on the lake side, and the others 16½ ft. high. They will be built mainly of 12x12-in. fir timbers, drift-bolted together and having the ends of the transverse ties dovetailed into the wall timbers. The bottom of each wall is a 12x18-in. footing timber backed by another 12x12 in. The cross-walls are about 8 ft. apart. At the corners of alternate cross-walls are posts, secured by scow bolts. The outer or lake side will have a sheathing of 3-in. white oak or maple plank placed vertically and secured by lagscrews. The top will have a deck of 6x12-in. planks 2 in. apart. The corners of the end cribs will be sheathed with steel plate.

A foundation of rubble stone will be laid in advance, at least two weeks before the crib is placed upon it. This will have a top width of 44 ft. with slopes of 1 on 1½, forming berms of 8 and 6 ft. on the lake and harbor sides of the crib. This foundation will be leveled before sinking a crib, so as to insure a continuous bearing for the longitudinal footing timbers. As soon as the crib is in place, it will be filled with stone to the top of the upper cross-ties, and riprap stone will be deposited along both sides to a height of 8 ft., with slopes of 1 on 1½. The superstructure or deck will be continuous over the cribs, and it will be filled with stone. The foundation will be of stones weighing from 50 to 200 lb. each, with not less than 10% weighing under 10 lb. For the upper 2 ft. of its thickness, however, the stones will weigh from 10 to 100 lb., to facilitate leveling the surface for the cribs. The filling will be of 3- to 120-lb. stones for the first five courses of the crib, and then stones weighing not less than 40 lb. each will be used. The riprap stones must weigh at least 200 lb. for the harbor side and 1,000 lb. for the lake side.

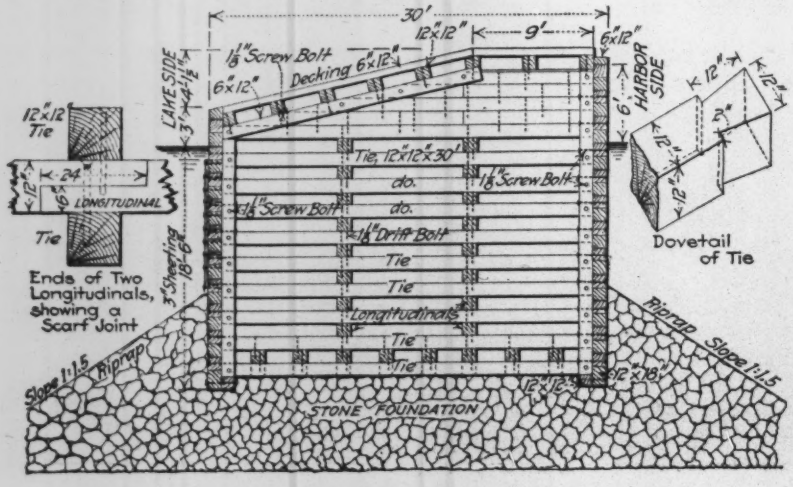


FIG. 1. CROSS-SECTION OF STONE-FILLED TIMBER-CRIB BREAKWATER AT CHICAGO

The east end of the breakwater, at the 400-ft. channel opening, will be protected by two clusters of 19 white-oak piles, wrapped with 1¼-in. wire cable. The piles will be about 6 ft. above the water, with bark removed to 1 ft. below the water. The tops will be rounded off and given three coats of hot carbolineum avenarius for 2 ft. from the top. These pile clusters will be surrounded by riprap of 1,000-lb. stones.

Rubble-Mound Breakwater

From the outer or southeast end of the present breakwater an extension will run due south for about 7,000 ft., with a 400-ft. opening opposite the mouth of the Chicago

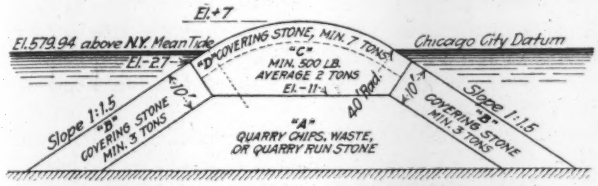


FIG. 2. CROSS-SECTION OF STONE RUBBLE-MOUND BREAKWATER AT CHICAGO

River. The present work, however, is for the first 2,270 ft. to this opening. The depth of water is from 27 to 31 ft.

This breakwater will have no cribs, but will be a mound of rubble stone approximately 125 ft. wide on the base, with side slopes of 1 on 1½ and a top radius of 40 ft., the crown being 7 ft. above the water. The cross-section is shown in Fig. 2.

The lower part of the core or central mass, extending to within 11 ft. of the water, will consist of quarry chips or waste, or quarry-run stone, with not more than 5% (in volume) of stone dust in any one load. The upper part of the core will consist of stones averaging 2 tons each and weighing not less than 500 lb., except for a limited amount of smaller stone to block and level the covering or outer course.

The sides of the core will be covered to a thickness of 10 ft. with stones weighing not less than 3 tons. This covering will extend to within 27 ft. of the water surface. The covering of the crown of the breakwater will be 5 ft. thick and composed of stones weighing not less than 7 tons each.

The stone for the two portions of the core (*A* and *C*, Fig. 2) may be placed by dropping or dumping it into the water, but the use of dump scows will be permitted only for the lower portion *A*, which includes the smallest class of material used on the work. The stones for the covering of the sides *B* and crown *D* must be set in place before being released. The former *B* must be placed before the upper part of the core *C* is deposited. The latter *D* must be placed as the core *C* is built up. All covering stones must be well supported on the underlying stones, the interstices being kept at a minimum for this class of work.

The amount of stone required per 1,000 ft. of breakwater in 29 ft. of water is estimated as follows: Lower core, *A*, 65,730 tons; side covering, *B*, 42,190; upper core, *C*, 24,810; crown covering, 14,460—total, 147,190 tons. This is based on an assumed average weight of 96 lb. per cu.ft. of stone and voids, with an allowance of 10 per cent. added for settlement and for stone outside of the normal section.

Volunteer Engineer Officers

Engineers who have been impressed with the "preparedness" arguments now being advanced so earnestly by those who think this country ought to be ready for war any moment will be interested in the recently amended requirements which must be complied with by candidates for commissions as officers of volunteer engineers. This amendment was issued by the War Department on Aug. 25, 1915, as General Orders No. 50.

After referring to the necessity for passing physical and moral examinations, and answering general questions as to experience, submitting documents in evidence before boards of officers of the Corps of Engineers and of the Medical Corps, United States Army, the order proceeds to give the qualifications for the grades of lieutenant and of captain. These are as follows:

3. Lieutenant—(a) The candidate must be an engineer in the active practice of his profession or in some business immediately connected with or concerned in engineering matters.

(b) He must either hold the grade of junior engineer, civil, electrical or mechanical, or higher grade in the civil service, or he must be a graduate from an approved engineering college, or belong to one of the following national engineering societies in a grade not lower than that shown opposite to each: The American Society of Civil Engineers—junior member; the American Institute of Mining Engineers—member; the American Society of Mechanical Engineers—junior; the American Institute of Electrical Engineers—associate. Or he must be or have been a noncommissioned officer of the grade of sergeant or above from one of the regular battalions of engineers.

4. Captain—(a) Same as 3 (a).

(b) He must either hold the grade of assistant engineer in the Engineer Department at Large or a corresponding engineer grade in the civil service in another department of the Government service, or be an associate member, or member of one of the national engineering societies enumerated under 3 (b), or have held a commission in the Corps of Engineers of the regular army.

(c) Sufficient knowledge of Field Service Regulations, Army Regulations and Tables of Organization to understand the type of information of which these regulations are the source.

5. Field Officer—(a) Same as 3 (a).

(b) He must either hold the grade of assistant engineer in the Engineer Department at Large or a corresponding engineer grade under the civil service in another department of the Government service, or be a member of one of the first three national engineering societies enumerated in 3 (b), or be a fellow of the American Institute of Electrical Engineers, or have held a commission in the Corps of Engineers of the regular army not more than two grades below that for which he desires to be listed.

(c) In addition to fulfilling the qualifications given in 5 (a) and (b), the candidate will be required to pass the examination prescribed herein for a field officer of infantry and an examination on the duties of engineer officers and troops in war, what they are and how they are performed.

Equal weights will be given to each of these examinations.

A Train-Ferry Steamer designed to break its way through heavy ice has been built for the Canadian government, to maintain communication across Northumberland Strait, between the mainland and Prince Edward Island. The distance is about 10 mi. The steamer is 285 ft. long and 52 ft. beam, and has a loaded draft of 18 ft. The sides are inclined at the water line, so that any pressure from the ice will tend to lift instead of to crush the hull. There are two tracks, and the stern of the ship is open to enable cars to be run in and out at the dock. For propulsion there are twin screws with triple-expansion engines aggregating 5,000 hp. A third screw under the overhanging bow is driven by a similar engine of 2,000 hp.; this serves to break up the ice by disturbing the water beneath it, and serves also to clear the ferry slip of ice. The six Scotch boilers are arranged beneath the train deck, with uptakes leading through the sides of the superstructure to four smoke-stacks, two on each side of the ship. The steamer has been built by Armstrong, Whitworth & Co., of Walker-on-Tyne, England.

Extraordinary Rain in St. Louis with Study of Runoff

By W. W. Horner*

In the afternoon of Aug. 19 the tail end of the Galveston storm reached St. Louis, and during the evening of that day and continuing up to the morning of the 21st the heaviest rainfall ever recorded in St. Louis occurred. The hourly intensities from one of the gages is shown by curve *A* and the mean intensities for three hours preceding by curve *B*. The intensity did not exceed an average of 0.9 in. in any one hour, but the total precipitation for the storm and for the 24-hr. period was nearly double any previous record.

The rain was accompanied by a high wind, generally from the north, which at one time reached a velocity of nearly 50 mi. per hr. The north walls of almost every building in the city were saturated, a great deal of plastering and decorating ruined and the outsides discolored by the ensuing efflorescence. Owing to the moderate intensity of the rain there was very little damage to the sewer system, one or two minor cave-ins of the older sewers occurring but no serious overcharge.

The total rainfall shown by a number of automatic gages is given in the accompanying table.

INCHES OF RAINFALL BY EIGHT ST. LOUIS GAGES

Station	Aug. 1-19	Aug. 19-20-21
Kingshighway and VonVersen	4.92	10.59
Shawmut and Ridge	4.17	10.54
Union and Patton	4.61	9.98
Forest Park	4.80	10.59
Kingshighway and Natural Bridge Rd.	4.83	8.54
Twenty-fifth and Mullanphy	3.59	8.12
Weather Bureau	3.67	7.39
Clayton	2.85	10.11

The first four gages shown are in the drainage area of the River des Peres within the city limits, and the Clayton gage is near the center of the watershed, about two miles west of the city limits. The Weather-Bureau gage is near the Mississippi River on top of a twenty-story building. All the other gages listed belong to the City of St. Louis and receive the attention of trained observers. The unusual number of automatic gages located closely together in the city is accounted for by the study that has been going on for some time of the variation in rates of rainfall from severe storms over small areas.

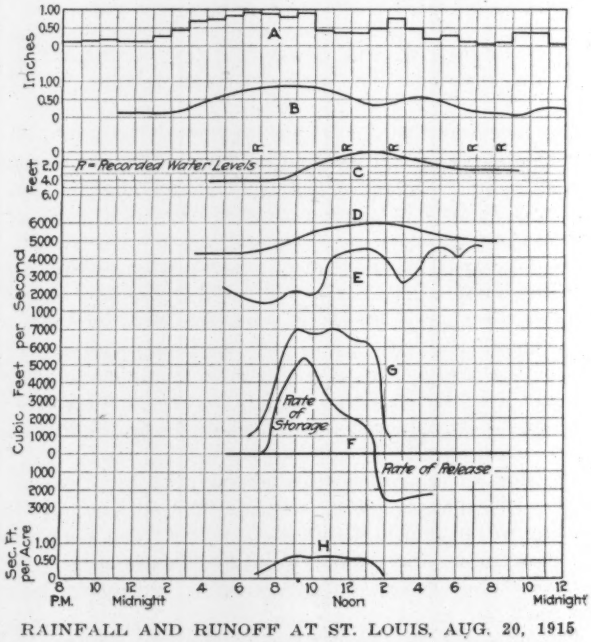
Owing to the general nature of the storm and the low intensity of rainfall, it is improbable that the variation in the totals recorded can be accounted for except by the very high wind, which at times drove the rain nearly horizontally. The five Western gages, which checked so closely at from 10 to 10.6 in. for the storm, are all on the roofs of two-story buildings and have some windbreak within a short distance to the north. Also, the registration of the automatic apparatus was quite closely checked by the amount of water in the reservoirs after the rain. There is very little reason to doubt, therefore, that the rainfall for the storm was approximately 10.6 in. and that for the 24-hr. period about 10.2 in.

St. Louis is unfortunately situated, however, in that along the whole crescent of its western limits it receives drainage from the adjoining county. The largest of the streams is called the River des Peres.[1] It has a total watershed of 70,000 acres, of which 16,000 is within the city limits. During the excessive rain of Aug. 20, this stream reached a stage far beyond any previous records. A brief description of some of the facts may be of interest.

Where the stream enters the city limits near Delmar Ave., the total drainage area from the county is approximately 10,000 acres. Three miles below, in Forest Park, it is crossed by a concrete-arch bridge carrying Clayton Road over it. This bridge has a span of 75 ft. and its arch crown is about 33 ft. above the stream bed. During most of the day of Aug. 20 Clayton Road was the only clear roadway across the stream, all other bridges, both highway and railroad, being flooded and submerged.

Between the point of entrance mentioned and Clayton Road, the stream receives the lateral drainage from 2,750 acres of sewered area and 2,240 acres of park and county area, making a total drainage area at Clayton Road of about 15,000 acres.

Early in the morning it was appreciated that the flood would exceed all records and that it would be advisable, if possible, to secure a measurement of the discharge,



RAINFALL AND RUNOFF AT ST. LOUIS, AUG. 20, 1915

but on account of the poor means of communication and because several bridges that had previously been selected as gaging stations were afterward found to be submerged, it was not until 6 p.m. that a party and the instruments were gotten together at the Clayton Road bridge, the only place where the whole flow was united in one stream. At that time, the water level had fallen from a high-water mark of 53.32 ft. above city datum to El. 51.

Current meter readings taken between 6 and 7 p.m. gave a mean velocity near the surface of 5.85 ft. per sec. Assuming a general average velocity of 4.6 ft. per sec., the discharge through the wetted area of 1,130 sq.ft. would be 5,200 sec.-ft., and it seems probable that the mean velocity at the flood stage must have been about 4.8 ft. per sec. and the discharge through an area of 1,242 sq.ft. about 6,000 sec.-ft.

Curve *C* shows the approximate stages of the stream below high water from 7 a.m. to 7 p.m. of the 20th. The points marked *R* are measurements made by the superin-

*Engineer of Design, Sewers and Paving, St. Louis, Mo.

[1]This stream enters at about the midpoint of the western limits, flows southeast through the main residence section and through Forest Park to a point about 1½ mi. east of the limits, then southwestwardly through a manufacturing district to the city limits again, at Lindenwood, where it is joined by a still larger branch from the county and follows around just inside of the limits of the city to the Mississippi River at the southern end of the town.

tendent at the park stables about half a mile above Clayton Road. The other points are estimated and were introduced for the purpose of the analysis which follows.

As there has for some time been a strong movement to have this stream inclosed as a sewer, it is of interest to compare the measured flow of 6,000 sec.-ft. with what might have occurred had that portion of the stream within the city been completely inclosed.

From high-water marks taken all along the stream and from the standard contour maps of the city, it has been estimated that at the time of the crest about 18,000,000 sq.ft. within the city was under water, most of this being in Forest Park. It has been further estimated, with much less accuracy, that at the beginning of the last sharp rise (that is, at 6:00 a.m., when the water was 4 ft. below the crest) about 14,000,000 sq.ft. was under water. These areas were prorated in accordance with the stages of the river shown on curve *C*, and from the mean flood area in each hour and the rise in that hour, the amount of water going into storage within the city for each hour is calculated. This amount is shown by curve *F*.

The actual flow at Clayton Road at the stage 2½ ft. below the crest had been measured and found to be 5,200 sec.-ft. The discharge at the crest had been calculated at 6,000 sec.-ft. The discharge at other stages within 4 ft. below the maximum was approximated and shown by curve *D*. From the discharges shown in curve *D* was then deducted the probable lateral drainage between the city limits and Clayton Road. This deduction consisted of the runoff from 2,750 acres of sewered area taken at from 80 to 90% of the previous hour's rainfall (an hour being approximately the critical time for the tributary sewers) and also of the runoff from 2,240 acres of park and county area taken at from 25 to 90% of the rainfall for the previous hour, the percentages being prorated at from 25 at 6 a.m. to 90 at 2 p.m. Subtracting these values from curve *D*, curve *E* was obtained. Of course this curve shows an entirely too sharp condition of rise and fall, as the storage in the county on the main stream, of which no account has been taken, would have tended to smooth out the discharge at the city limits and the storage in the sewered and park areas would have been sufficient to balance with curve *D*. For the purposes of this analysis, no attempt was made to assume the actual form that curve *E* should have had.

We have then from curve *E* the amount of water passing Clayton Road due to the runoff from the county, and from curve *F* the amount of runoff from the county that entered the city but was used up in storage before reaching Clayton Road. The summation of the ordinates of these two curves produces curve *G*, which is an approximate theoretical discharge of the county drainage area, and dividing the points on this curve by the acreage of 10,000 gives curve *H* as the approximate runoff per acre of the county area—the maximum being 0.64 sec.-ft.

The analysis given is of course highly speculative and presents no claims to accuracy, as the data on the rate of rise are crude and the discharges, approximately from one measurement, probably missed many variations of importance. It seems safe to say, however, that the maximum discharge from the rural area must have been at least 0.5 sec.-ft. per acre.

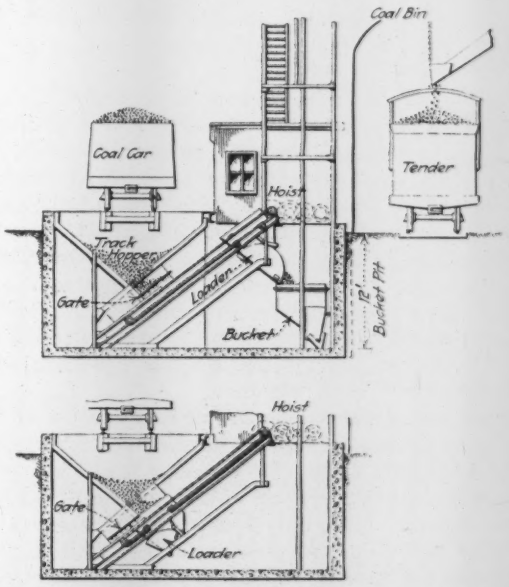
Curve *B* shows the mean intensities for 3-hr. periods, which is approximately the critical time at Clayton Road. Comparing the 0.5 already given with the maximum 3-hr. intensity of 0.85 shows a percentage of runoff of at least 60, which is higher than engineers have been wont to assume for a drainage area of this character and size.

Another interesting point in the demonstration is the evidence that had the storage been eliminated, the crest would have occurred at about 10 o'clock, which is the time indicated by the intensity curves.

Shallow-Pit Coaling Station

A new design of coal-handling plant for railway coaling stations of the elevator-bucket type has a bucket pit only 12 ft. deep. This reduces the cost of the underground work and may preclude trouble with water which would be encountered with the deeper pit usually required where the coal is fed by gravity from the track hopper to the elevator bucket.

The special feature, as shown in the accompanying drawing, is that the coal is fed through an undercut roller



LOCOMOTIVE COALING STATION WITH SHALLOW PIT AND MECHANICAL LOADER

gate to a traveling loader, or bucket. This loader runs up inclined guides and dumps the coal into the elevator bucket, which in turn delivers it to the elevated bin in the usual way. The loader is of 2½ tons' capacity. It is geared to the automatic electric hoist that operates the elevator bucket, and the movement of the loader effects the opening and closing of the gate. This coaling plant is manufactured by the Roberts & Schaefer Co., of Chicago.

A Large Coal-Shipping Pier for the Baltimore & Ohio R.R. is to be built at the Curtis Bay terminal, at Baltimore. It will be of steel construction, 700 ft. long and 115 ft. wide. Two car-dumping machines at the shore end will deliver the coal to belt conveyors leading to movable towers on the pier, from which it can be delivered to vessels on either or both sides of the pier. The car-dumpers will be able to handle 40 cars per hr. (50-ton cars). The conveyor belts will be run at speeds of 250 to 500 ft. per min., giving a capacity of 3,000 to 6,000 tons per hr. The total shipping capacity will be about 10,000,000 tons per yr. The pier was designed by Francis L. Stuart, Chief Engineer. Contracts are to be let soon, and it is expected that 18 months will be required for the construction. The cost will be about $1,500,000.

Construction and Operation of Gloversville Sewage Works--I

By H. P. Eddy* and H. J. Hanmer†

SYNOPSIS—The design, construction and cost of a 3,000,000-gal. sewage-treatment works are here given, and the operating results will be summarized in a second article. The works consist of coarse screens, primary settling tanks, fine screens, a dosing tank, trickling or sprinkling filters, secondary tanks, final sand filters and sludge pumps and beds.

Construction and Cost

This and a following article are designed to give, not only the main features of the design and construction of an unusually complete sewage-treatment plant, but also a summary of its cost and, more unusual, a review of several years' operation.

In 1910 Gloversville had a population of 20,642, about 32 mi. of sanitary and 5 mi. of storm sewers, besides 3 mi. of 15- to 28-in. intercepting and outfall sewer. A large portion of the sewage consisted of troublesome wastes from 20 to 30 tanneries. From 1899 to 1909, about 70 lawsuits due to the pollution of Cayadutta Creek were brought against the city and the mill owners. In 1908-9, experiments on the treatment of the sewage were conducted.[1] As a result of these experiments and of other studies, it was decided to treat the sewage by screening, sedimentation, sprinkling filters, secondary sedimentation and intermittent sand filtration. A high degree of treatment was considered necessary, both because of the character of the sewage and the fact that the dry-weather flow of Cuyadutta Creek at the sewage works was sufficient to dilute but one-third of the volume of sewage, the drainage area being only 14.2 sq.mi. It was also decided that the mill owners must install and maintain settling tanks for the removal of suspended matter from their wastes before the latter were discharged into the city sewers.

Works to treat a daily average of 3,000,000 gal. of sewage, with conduits for a flow of 10,000,000 gal. a day, were designed in 1909 by Metcalf & Eddy, Boston, Mass. The works were put in operation on Dec. 1, 1912.

Preliminary Screening and Settling

Screen Chamber—The sewage discharged from the trunk sewer passes into a screen chamber 12 ft. square in plan, containing a screen made of ⅜x1¾-in. wrought-iron bars spaced 1½ in. c. to c., inclined at an angle of about 60 deg. to the horizontal. The effective area of the screen under ordinary operating conditions is about 15 sq.ft. and the total area is 84 sq.ft.

Primary Settling Tanks—Each of the two primary settling tanks (Figs. 1 and 2) is of reinforced concrete, circular in plan, 36⅓ ft. in diameter at the top of the cylindrical portion and 35 ft. in diameter at its base. The inverted conical bottom is 18½ ft. deep, with side slopes of 1 to 1. The depth of sewage in the tanks under normal operating conditions is about 49 ft. The sewage enters each tank at the top through a wooden trough to a wooden down pipe 3 ft. in diameter, and flows down and out under the edge of a distribution cone, also of wood. The sewage then passes up about 31 ft. to the wooden collectors, or outlet troughs, which are distributed over the surface in order to draw as nearly as practicable a uniform flow from all parts of the tank. The sides of the wooden distribution cone in the west tank are built on a slope of 45 deg. and those in the east tank on a slope of 60 deg., which latter slope has been found to be the more satisfactory. From the collector troughs the sewage goes to the dosing tank for the sprinkling filters.

The total capacity of each tank below the flow line is approximately 268,500 gal. Based on the assumed sewage flow of 3,000,000 gal. per day the velocity of sewage in the vertical tanks is 8.7 ft. per hr. and the time of passing through the tanks, 3½ hr.

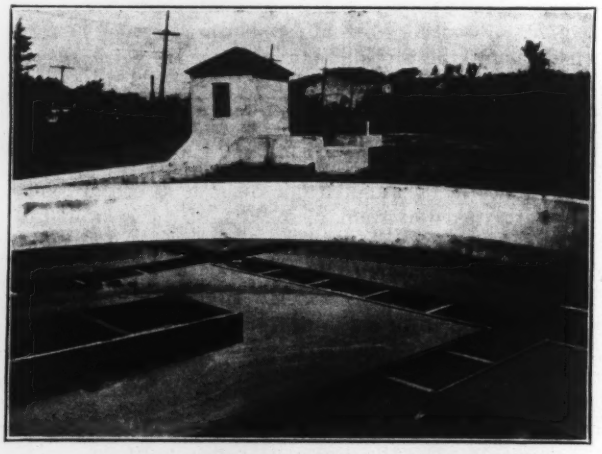


FIG. 1. PRIMARY SETTLING TANK, GLOVERSVILLE, N. Y.

The excavation for these tanks was in gravel, sand and blue clay, with a little hardpan near the bottom. An area 85 ft. long and 47 ft. wide was excavated, using 10x10-in. bracing with 2-in. sheeting. The groundwater encountered required a large pumping equipment, consisting at first of two 6-in. pulsometer pumps and finally of two 8-in. centrifugal pumps. In addition to these, two 3-in. diaphragm hand pumps were operated, using four men to each pump, to lift the water from the bottom excavations 6 or 8 ft. to the pulsometers. The material was shoveled into a 1-yd. skip and then hoisted to the surface by a derrick. The concrete (1 part cement, 6 parts aggregate, hand-mixed) was placed directly on screened gravel, and wrought-iron pipes were inserted at several points in the conical bottoms to serve as drain pipes during the construction work. After the concrete in the conical bottoms had hardened, these pipes were used as

*Consulting Engineer, 14 Beacon St., Boston, Mass.
†City Engineer, Gloversville, N. Y.

[1] A report on these experiments was noticed in the "Engineering Literature Supplement" of "Engineering News," Dec. 15, 1910, p. 63.—Editor.

pump wells for the groundwater coming in through the screened gravel under the concrete and the wrought-iron drain pipes.

Fine Screens—The settled effluent passes in succession through fine screens of ½-, ¼- and ⅛-in. mesh, set in the channel leading to the dosing tank.

Dosing Tank—The dosing tank is approximately 13x24 ft. in plan, with a sewage depth of 3½ ft. (see section, Fig. 3). A 30-in. Pacific automatic dosing siphon discharges the tank automatically into a conduit leading to the trickling filters.

Trickling Filters—The trickling filters have reinforced concrete walls and floors. Their surface area is 3.07 acres, divided into three equal units. The main distribution conduit is in the center of each filter unit.

The side walls are 6 in. thick and 5½ ft. high, reinforced every 12 ft. with pilasters, which also support columns for the roof beams. The outside columns are grooved to receive side planks. The main roof beams are of reinforced concrete, 8x15½ in. in section, 12 ft. c. to c., with their tops 6½ ft. above the surface of the filter stone. The stay beams running at right angles to the main roof beams are 5x10½ in. in section, 24 ft. c. to c. The beams and columns (Fig. 4) support the wooden plank covering that is put over the filters during the winter months.

Distribution System—From each main distribution conduit there are 31 lateral distributors (Fig. 5), 12 ft. c. to c., on each side of the main distributor. These distributors are 6-in. vitrified pipe incased in concrete, and from

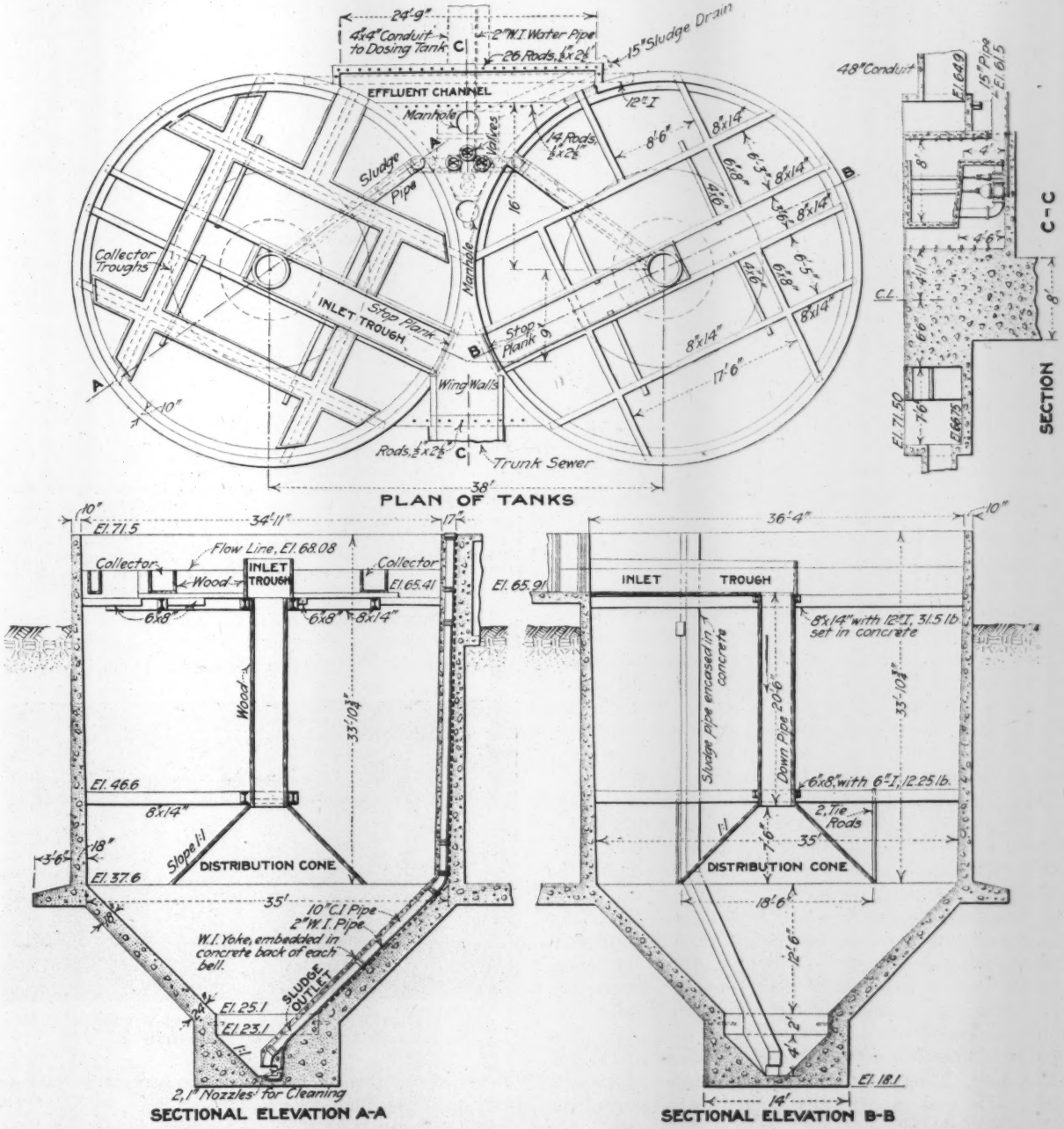


FIG. 2. PLAN AND SECTIONS OF PRIMARY SETTLING TANK, GLOVERSVILLE SEWAGE WORKS

them 3-in. cast-iron risers, 12 ft. c. to c., extend to Taylor four-lobed square spray nozzles. There are 930 nozzles in the three units, or approximately 303 per acre, spaced 12 ft. c. to c. in each direction, with the plane of the nozzle orifice set about 8 in. above the surface of the filter stone. The maximum static head on the nozzles is 6.34 ft.

Crushed Stone—The 23,146 cu.yd. of filtering medium is 1- to 2½-in. crushed and screened limestone, from Mayfield, N. Y. The stone was delivered at the filter plant over a spur track to an unloading platform and discharged directly into dump cars of 1 cu.yd. capacity run underneath the platform, and thence hauled to any part of the filters over temporary narrow-gage tracks. The depth of stone in the filters ranges from 4½ to 4⅔ ft. and averages 4 ft. 7½ in.

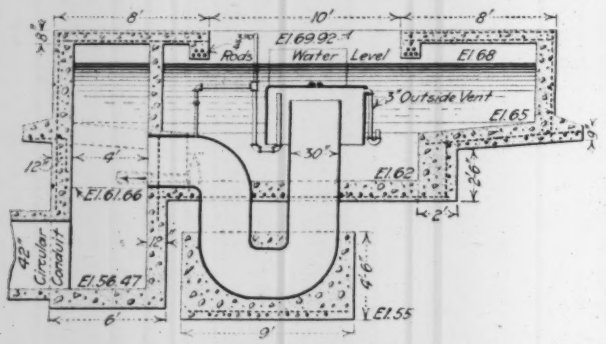


FIG. 3. SECTION OF DOSING TANK, GLOVERSVILLE SEWAGE WORKS

Drainage System—The floor system consists of a 3-in. concrete floor on which 6-in. vitrified slotted half-round undercut tiles were placed with spaces of 1 in. between the ends and 3-in. between rows. The concrete floor was laid on a slope of 1 in 225 from the flushing gallery, in the center of each filter unit, to the outer edge where the underdrain is located. The half-round tiles are intended to support the filter stones, leaving free waterways for the filter effluent and providing space for ventilation. The distance from the flushing gallery in either direction to the main underdrain is approximately 60 ft.

The flushing gallery (Fig. 5) is 3½ ft. high by 2½ ft. wide, and is built directly over and as a part of the structure containing the main distribution conduit located in the center of each of the three filter units, the gallery and conduit being separated by a 6-in. reinforced-concrete floor, the top of which is flush with the top of the concrete drainage floor on either side. There are portholes through the side walls on the top of this floor opposite the end of each line of floor tiles, for the purpose of flushing out obstacles in the drainage system.

The main underdrains, located on each of the long sides of each filter unit, are from 1 to 2 ft. wide and from 6 to 15 in. deep. They are covered by 3-in. reinforced-concrete slabs. The main underdrains slope 1 in 500 toward the inlet end of the filters.

Treatment of Sprinkling-Filter Effluent

Secondary Settling Tanks—There are two secondary reinforced-concrete settling tanks of much the same type as the primary settling tanks already described. Their total depth is 33¼ ft. The cylindrical portion of each tank is 35 ft. in diameter at the base and 36 ft. at the top. The inverted conical bottoms are the same as those for the primary settling tanks. The total capacity of each tank below the flow line is 121,000 gal.

On the basis of an average sewage flow of 3,000,000 gal. per day, the detention period in the secondary tanks is 1.2 hr. and the vertical velocity 8.7 ft. per hr.

The thickness of the shell of the tanks at the base of the cone is 20 in., at the base of the vertical side wall, 15 in., and at the top of the side wall, 9 in. These tanks were constructed after the primary tanks, the same braces and forms being used again as far as possible. Practically the same kinds of soil were encountered, but the groundwater was not so troublesome because of the lesser depth.

Final Treatment of Effluent

Sand Filters—There are three intermittent sand filters, having a total effective sand area of 2.72 acres. The excavation for these filters varied from 1 ft. to 12 ft. in depth and was practically all done with pick and shovel, most of the excavated material being hauled out in dump wagons, although some of the top soil, which was rather loamy, was hauled out with horse scrapers and some with dump cars. There is a 15-in. vitrified pipe main underdrain in each filter, and 6-in. lateral underdrains, all laid with open joints, the upper half of the bells being removed and the pipes surrounded with two layers of graded crushed stone. The filter sand is 3 ft. deep and was furnished under the following specifications:

> The effective size shall not be less than 0.25 nor more than 0.35 mm. The uniformity coefficient shall not be more than 3. The sand shall not contain more than 1 per cent. finer than 0.13 mm. and shall be entirely free from particles over 5 mm. in diameter. The sand shall consist largely of quartz grains and shall not contain grains liable to disintegration. It shall be free from loam, clay and dust, or perishable matter, and shall, if necessary to render it clean and of proper size, be washed and screened before it is placed in the filters.

The sewage is distributed over the surface of each sand filter by a trough with concrete bottom and wooden sides. Adjustable side openings in the carrier make it possible to control the quantity of sewage discharging to various portions of the filter surface.

FIG. 4. GLOVERSVILLE TRICKLING FILTERS AND WINTER-ROOF SUPPORTS

Sludge-Pump Well—The sludge flows by gravity from the primary and secondary settling tanks to the sludge-pump well, which is a concrete structure 12x11½ ft. in plan and 17 ft. deep. Here the sludge is lifted about 15 ft. by a motor-driven centrifugal pump and discharged through 1,065 ft. of 15-in. vitrified pipe to the sludge-drying beds.

The pumping equipment comprises a Goulds No. 6 single-stage vertical centrifugal pump of 1,000 gal. per

min. capacity, driven by a 10-hp. Westinghouse motor, 850 r.p.m., operating on 60-cycle two-phase 220-volt alternating current. The pump is protected by a flat bar screen of 3/8x1¾-in. bars on edge, spaced 1½ in. c. to c.

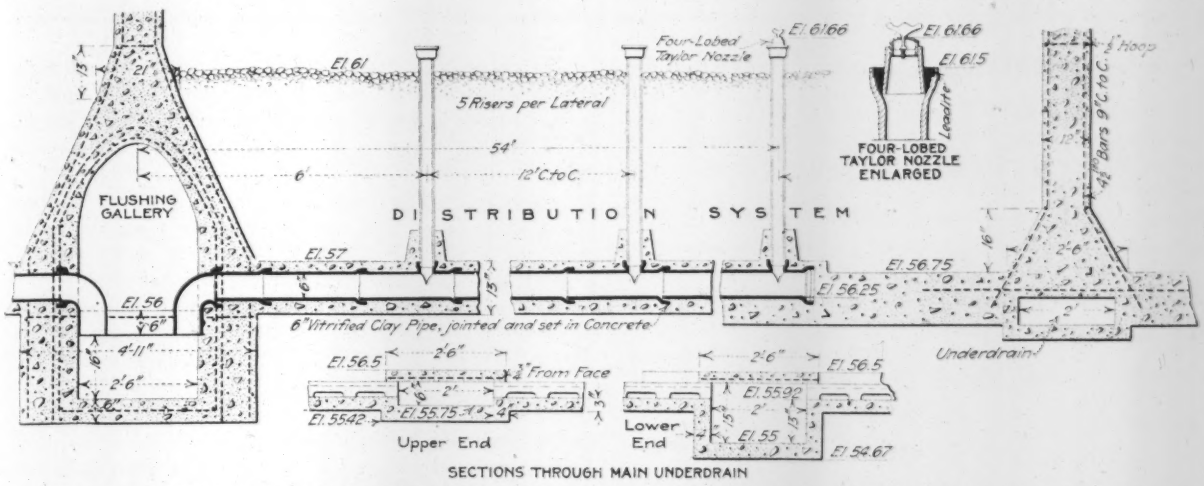


FIG. 5. DETAILS OF DISTRIBUTION AND UNDERDRAINAGE SYSTEM, GLOVERSVILLE SEWAGE WORKS

Sludge Beds—There are four sludge beds, having a total effective sand area of 2.63 acres, the construction of these beds being much the same as that for the sand filter beds. The specifications for the sand, however, are somewhat different:

> The sand of which the sludge beds will be composed shall have an effective size of at least 0.15 mm. and a coefficient of uniformity not exceeding 10. The sand shall be free from clay, loam, or other foreign substances which would render it unsuitable for the use for which it is provided, and it shall not contain material liable to disintegration under ordinary weather conditions.

The main underdrain is 10 in. in diameter with 6-in. lateral underdrains branching from it. The sand is 15 in. deep at the center and 12 in. deep at the edge of the beds next to the embankments.

Loss of Head—The total head lost in the plant is shown by Table 1.

Unit Prices—Table 2 gives some of the main items of work with the unit price of each, and Table 3 summarizes the cost by elements of the plant.

TABLE 1. LOSS OF HEAD IN SEWAGE WORKS

	Elevations	Fall in Feet
Elev. crown trunk sewer at screen chamber	69.00	....
Elev. flow line in primary settling tanks...	68.08	0.92
Max. high water line in dosing tank.....	68.00	0.08
Elev. orifice of nozzles in trickling filters...	61.66	6.34
Elev. flow line in secondary settling tanks..	55.00	6.66
Surface of lowest sand filter bed.....	50.00	5.00
Elev. crown of lowest 20-in. outlet to creek	47.27	2.73
Total fall		21.73

TABLE 2. PRINCIPAL ITEMS AND CONTRACT UNIT PRICES, GLOVERSVILLE SEWAGE WORKS

Earth excavation:	
In trench depth 0-8 ft., per cu.yd.....	$0.45
In trench depth 8-14 ft. per cu.yd.....	0.75
New channel for creek and tailrace, per cu.yd.....	0.39
Deepening present creek channel, per cu.yd.....	0.49
For primary and secondary settling tanks, per cu.yd.:	
Depths 0-10 ft.....	0.75
Depths 10-20 ft.....	1.25
Depths 20-30 ft.....	1.75
Depths 30-40 ft.....	2.50
Depths 40-50 ft.....	3.00
For sprinkling filters, per cu.yd.....	0.39
For sludge beds and sand filter beds, per cu.yd.....	0.39
Below masonry or below grade, per cu.yd.....	1.00
Concrete masonry, per cu.yd.:	
In primary and secondary settling tanks.....	9.25
In sprinkling filter (excl. of floors).....	8.25
In other structures.....	10.00
Sprinkling filter floor system (inc. reinforcement), per sq.yd.....	2.25
Steel reinforcing bars for concrete, per lb.....	0.025
Steel channels for supporting reinforcing bars, per lb.	0.03
Steel I-beams, per lb.....	0.025
Steel mesh reinforcing metal, per lb.....	0.065
Gravel filling below grade, per cu.yd.....	1.50

Pipe Laying per Linear Foot (exclusive cost of pipe)

Size of Pipe, In.	Vit. Pipe with Joints of—Cement Mortar	Vit. Pipe with Joints of—Sulphur Sand or Bituminous Compound	Cast-Iron Pipe with Lead Joints	Vit. Pipe Underdrain Including Excavation and Crushed Stone
4	....	....	$0.15	$0.10
6	$0.05	$0.07	0.20	0.12
8	0.07	0.10	....	0.15
10	0.09	0.13	0.30	0.17
12	0.11	0.16	....	0.20
15	0.14	0.21	....	0.21
18	0.20	0.26	....	....
20	0.22	0.29	....	....
24	0.26	0.35	....	....
28	0.30	0.41	....	....

Laying, jointing and surrounding with concrete 6-in. vit. pipe distributors, per lin.ft.....	$0.25
Sand furnished and placed in sludge beds and sand filters, per cu.yd.....	0.90
Placing, jointing and setting risers in sprinkling filters, per riser.....	0.60
Crushed stone for trickling filters (1 to 2½-in.), per cu.yd.	1.75

TABLE 3. ITEMIZED COST OF SEWAGE WORKS

	Total Cost	Unit Cost
Screen-chamber substructure...	$445	
Screen-chamber house (total exterior volume 1,350 cu.ft.)....	535	$0.40 per cu.ft.
Primary settling tanks (total capacity both tanks 537,000 gal.)	15,761	26.10 per 1,000 gal.
Dosing tank (total capacity 8,800 gal.)	1,404	160.00 per 1,000 gal.
Trickling filters (3.07 acres stone area)	106,560	34,700.00 per acre
Secondary settling tanks (total capacity both tanks 242,000 gal.)	9,292	38.40 per 1,000 gal.
Sludge beds (2.63 acres effective sand area)	9,097	2,450.00 per acre
Sand filter beds (2.72 acres effective sand area).....	24,717	9,090.00 per acre
Sludge-pump well (total capacity 16,230 gal.).....	1,233	75.90 per 1,000 gal.
Sludge-pump house (total exterior volume 1,880 cu.ft.)...	535	0.39 per cu.ft.
Sludge-pumping machinery	460	
Conduits and pipe lines (sewage, effluent, sludge, water).....	11,759	
Grading, drives, walks, trees, cleaning up, etc.....	3,801	
Creek deepening and straightening	1,396	
Miscellaneous	58	
Extras, claims, incidentals, delays and damages.....	1,700	
Total cost	$188,754	

On the basis of a daily capacity of 3,000,000 gal. the cost of the plant was $62,900 per million gallons of sewage treated per day.

The construction work was under the general supervision of City Engineer Morrell Vrooman, with Assistant City Engineer Harry J. Hanmer in immediate charge of construction. The consulting engineer was Harrison P. Eddy, of Metcalf & Eddy, Boston, Mass. The contractor was Albert L. Willey, Binghamton, N. Y.

Field and Office

Quebec-Bridge Camp and Yards

The arrangements for storing materials and housing workmen and engineers on a contract as great as the Quebec Bridge are always of interest. A single camp was built for the men working on both sides of the river, although, as it seems now, sufficient accommodations could have been secured without any camp whatever.

The arrangement of bunkhouses, dining hall, office, bungalows, etc., is shown in Fig. 3. Water-supply lines (2 to 4 in.) and sewers (6 to 10 in.) were run to the various buildings. The camp layout was shown in detail in the issue of Mar. 5, 1914, p. 499.

The crane runways for the storage yards are at right angles to the railroad tracks. A single crane is used on each. (This is quite different from the arrangement of the Phoenix company during the work on the old bridge—two cranes on a very long runway parallel to the railway.) The columns of the runway are staggered, Fig. 1, so that members longer than the crane span can be shifted along the runway by traversing the trolleys back and forth as the crane runs along.

Each yard was designed for storing all the members of an anchor arm at the same time, to provide for the chance of delay in erection. The principal piling arrangements are shown in the typical yard sections. To minimize rehandling, only those members which are to be used at about the same time are piled on one another. All members are placed with regard to their ultimate position on the bridge, so that they do not need turning end for end. At least 1 ft. clearance is maintained between adjacent pieces.

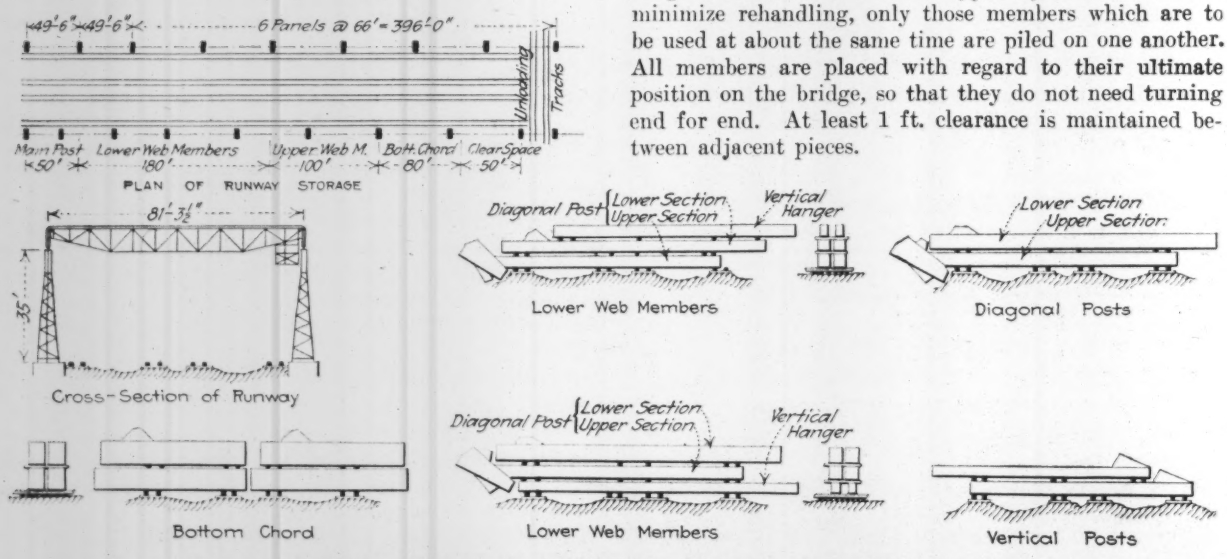


FIG. 1. STORAGE YARD AND TYPICAL PILING, NORTH SIDE, QUEBEC BRIDGE

FIG. 2. NORTH SHORE MATERIAL YARD, QUEBEC BRIDGE

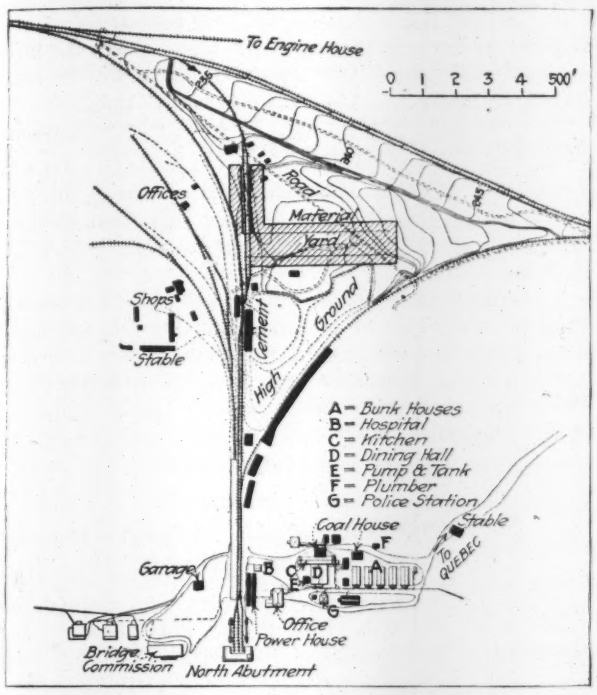


FIG. 3. CAMP AND NORTH SIDE YARD

Pieces with outstanding angles are generally handled with hooks, all others with wire-cable slings. For many parts special handling devices were built. The maximum amount stored at one time on the north side has been 10,000 tons and on the south side 12,000 tons.

The crane runway on the north side, patterned after one in the shops of the St. Lawrence Bridge Co., has box girders to carry the crane track. The runway for the south side has plate girders whose top flange is stiffened by a horizontal lattice-girder bracketed out from the girder web.

Each side of the river has its own electric-power station (or rather, substation; the two are fed from separate sources of energy) and compressor plant. The north-side station contains two 250-kw. motor-generator sets (one regular, one spare) to convert 2,200-volt alternating current to 220-volt direct current for the traveler. The primary supply is received at 22,000 volts. There are also four 100-hp. motor-driven air compressors, delivering air at 100 lb. gage pressure. The south-side plant is similar, but has only three 100-hp. compressors. The incoming current here is received at 11,000 volts.

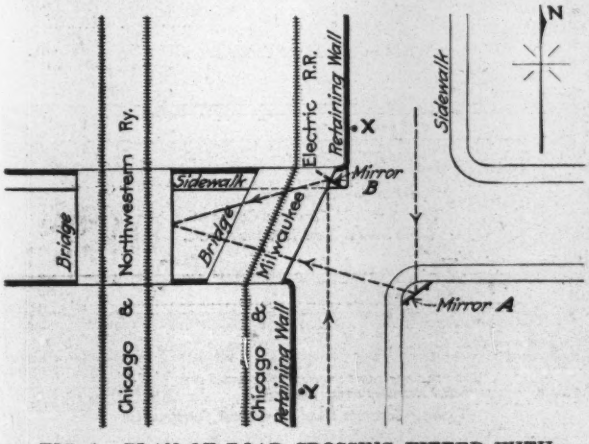


FIG. 1. PLAN OF ROAD CROSSING FITTED WITH MIRRORS, GLENCOE, ILL.

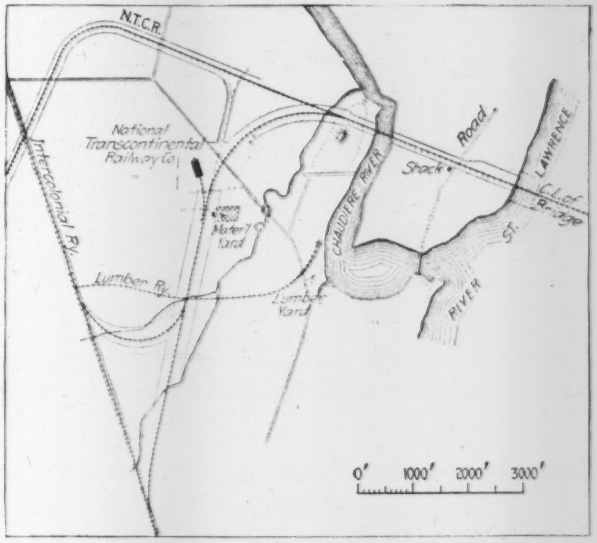


FIG. 4. SOUTH SIDE YARD, QUEBEC BRIDGE

Mirrors at Road Intersections

The use of mirrors for the protection of highway traffic at road intersections where the view is obstructed was noted in *Engineering News*, Aug. 5, and reference was made to an installation at Glencoe, Ill. The situation at this point is shown clearly by the accompanying plan and views, for which we are indebted to Robert A. Allton, Assistant Engineer of the Sanitary District of Chicago.

It will be seen that there is a jog in the street lines at the intersection, while the view is obstructed by the railway fills and bridge abutments.

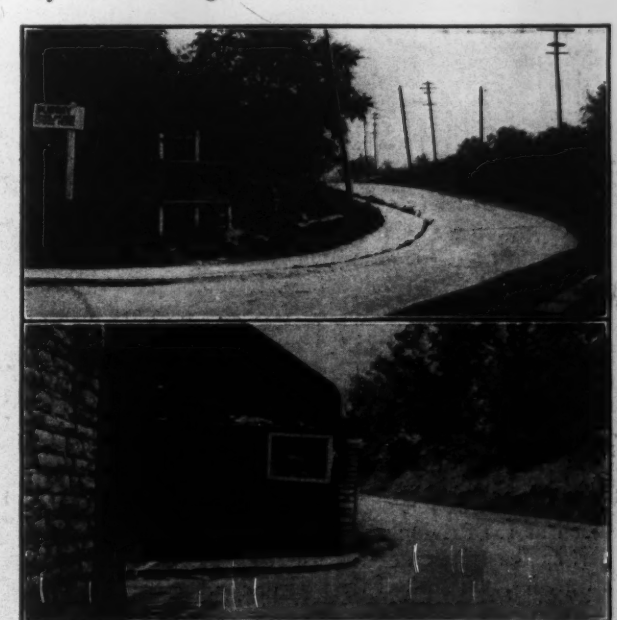


FIG. 2. MIRRORS AT ROAD CROSSING AT GLENCOE, ILL.

Upper view looks south from point X on the plan, showing mirror A. Lower view looks north from point Y, showing mirror B under the railway bridge

Hump-Yard Calculations

By Charles C. Wentworth*

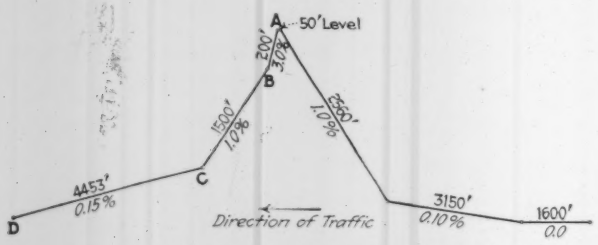
A simple formula expresses the motion of cars when drifting either on level or on up- or down-grades:

$$S_2^2 = S_1^2 + 30\,GL - 1\tfrac{1}{2}\,RL$$

in which

S_1 = speed at beginning of distance L, mi. per hr.;
S_2 = speed at end of distance L, mi. per hr.;
G = rate of grade in ft. per station (uniform for distance L);
L = length of track, in 100-ft. stations;
R = rolling friction, lb. per ton.

Evidently GL is the rise or fall in feet from one end to the other of the stretch of track under consideration. It



HUMP-YARD GRADES

is positive when the train is coasting downhill, negative when drifting uphill.

In this formula the weight of a car is immaterial, except as it may govern in selecting a value for R, because the formula applies equally to each single ton.

A gravity hump yard is to be built to the profile sketched, in a nearly level country. The accelerating grade is from A to B; the ladders lie in most part from B to C, and the tracks for classified cars from C to D.

Assuming an initial speed at A of 2 mi. per hr., which will pass about 250 cars per hour, the following figures may be made from the formula. The assumed values for R, as given, are probably too great, but may be met with on some cars.

	S_1	S_1^2	G	L	R	S_2^2	S_2	Speeds, mi. per hr.
A to B	2.0	4.0	3.0	2.0	10	154	12.4	A = 2.0, B = 12.4
B to C	12.4	.154	1.0	15.0	12	334	18.3	B = 12.4, C = 18.3
C to D	18.3	334	0.15	44.53	8	00	00	C = 18.3, D = 0.0

The previously unknown quantity found by each operation is set in heavy type, the others being known or, in the case of R and the final stoppage at D, assumed.

A train of 100 cars pulling in on the approach to the hump would weigh 7,300 tons, including one road engine, and its length would be 4,100 ft. We have then for its resistance of gravity:

1,600 ft. of train on a level grade = 2,800 tons, which at 8 lb. per ton has a resistance of	22,400 lb.
2,500 ft. of train on a 0.1% grade = 4,500 tons, which at 10 lb. per ton has a resistance of	45,000 lb.
Total	67,400 lb.

In the foregoing the engine is included as part of the train both in weight and length, and the tractive force is taken at the driving-wheel contact, not back of the tender, as the traction at the wheel contact is constant for all grades. The wheel traction of the engine under consideration, at 20% of the driving-wheel weight, is 67,460 lb. The train therefore can pull in with its engine ahead.

*Principal Assistant Engineer, Norfolk & Western Ry., Roanoke, Va.

To go over the hump the head engine can be cut off and two pusher engines put in the rear. When the first car reaches the summit of the hump the conditions are:

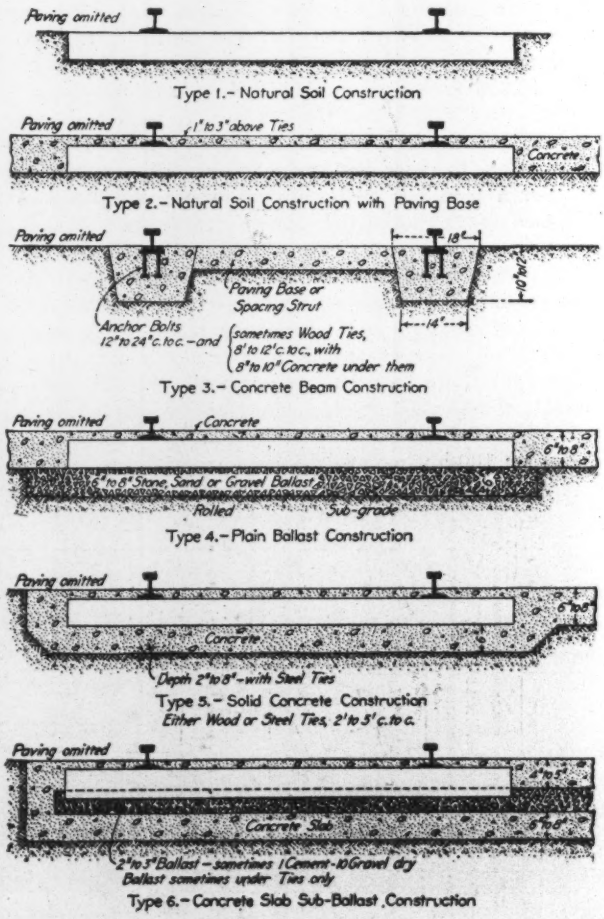
Length of train and two engines, 4,200 ft.; weight, 7,600 tons. Of this, 2,560 ft. of train, or 4,480 tons, would be on a 1% grade which at 28 lb. per ton calls for a traction at rails of	125,550 lb.
The rest of the train and engines are on a grade of 0.1%, or 3,120 tons at 10 lb.	31,200 lb.
Total	156,640 lb.

The rail traction of the two pusher engines under consideration, at 21% of the weight on drivers, is 158,256 lb.

The same formula can be used to determine the rolling friction of a train per ton when starting from rest on a downgrade and achieving a certain velocity in a known distance. If it should be desired to find how far a car will go up a grade with a known initial velocity, it should be noticed that in this case G is negative. On a level, as G is zero, the term containing it drops out.

Street-Railway Track in American Cities

A compilation of street-railway track constructions employed in 41 American cities of over 100,000 population has recently been made by the United Railways and Electric Co., of Baltimore, and is included in the report of the Committee on Way Matters of the American Electric Railway Engineering Association, presented at the


Paving omitted
Type 1.- Natural Soil Construction
Paving omitted
1" to 3" above Ties
Concrete
Type 2.- Natural Soil Construction with Paving Base
Paving omitted
18"
Paving Base or Spacing Strut
Anchor Bolts 12" to 24" c. to c. - and
sometimes Wood Ties, 8' to 12' c. to c., with 8" to 10" Concrete under them
14"
Type 3.- Concrete Beam Construction
Paving omitted
Concrete
6" to 8"
6" to 8" Stone, Sand or Gravel Ballast
Rolled Sub-grade
Type 4.- Plain Ballast Construction
Paving omitted
Concrete
Depth 2" to 8" - with Steel Ties
Type 5.- Solid Concrete Construction
Either Wood or Steel Ties, 2' to 5' c. to c.
Paving omitted
Concrete Slab
2" to 3" Ballast - sometimes 1 Cement - 10 Gravel dry
Ballast sometimes under Ties only
Type 6.- Concrete Slab Sub-Ballast Construction

GENERAL TYPES OF STREET-RAILWAY TRACK USED IN AMERICAN CITIES

San Francisco convention Oct. 4-8. The total of 7,331 mi. of track is divided among five types of construction (shown by the drawing herewith) as follows:

Type 2	23%	1718.1 mi.
Type 3	2%	142.5 mi.
Type 4	47%	3415.9 mi.
Type 5	21%	1531.4 mi.
Type 6	3%	253.9 mi.
Modification types 3 and 4	4%	278.8 mi.

If types 3, 5 and 6 are taken together as "concrete-ballast construction," they give 26% of the total mileage. This is in general agreement with the statement made by the same committee a year ago, that 20% of roads reporting to it used concrete ballast. The various types are shown in the accompanying sketch. The many varied details of drainage, rail chairs, spikes, cross-braces, nose blocks and paving are necessarily omitted.

Smoothness-Testing Machine for Pavements

Prof. R. D. Kneale of the department of civil engineering of the Georgia School of Technology proposes that paving specifications include a clause covering smoothness of the finished surface. In connection with his studies of street and road pavements in Fulton County and Atlanta, Ga., he has constructed the smoothness-testing machine shown in Fig. 1.

The machine consists of two steel-shod parallel runners, rigidly braced far enough apart to support between them the working mechanism shown in Fig. 2. The instrument weighs about 70 lb. and can be handled by one man. It folds to 6 ft. in length, which is convenient for transportation in automobile, wagon or train.

The gear *L*, held in contact with the pavement by a spring, drives by shaft *H* and bevel and worm gears a recording-paper drum *E*, usually at the rate of 1 in. for 8 ft. of pavement. On the paper two curves are traced, one representing the vertical motion of the gear *L* itself, while the other represents the vertical motion of a steel arc *P*, which is intended to represent a wagon-wheel rim. Both *P* and *L* communicate their vertical oscillations to two horizontally moving pencil levers *C*, arranged with brass points *D* to mark on the drum *E*. Their motions are also registered on two dials *A*, whose reading gives at each moment the total vertical rise (or fall) of *L* or *P*.

The manner of registering is as follows: When arc *P* goes over a hump it rises against the spring pressure;

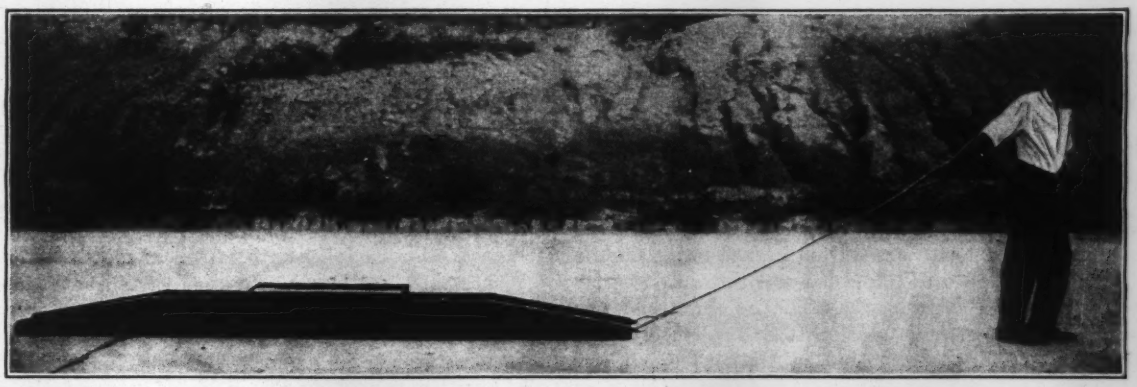


FIG. 1. KNEALE'S SMOOTHNESS-TESTING MACHINE FOR PAVEMENTS

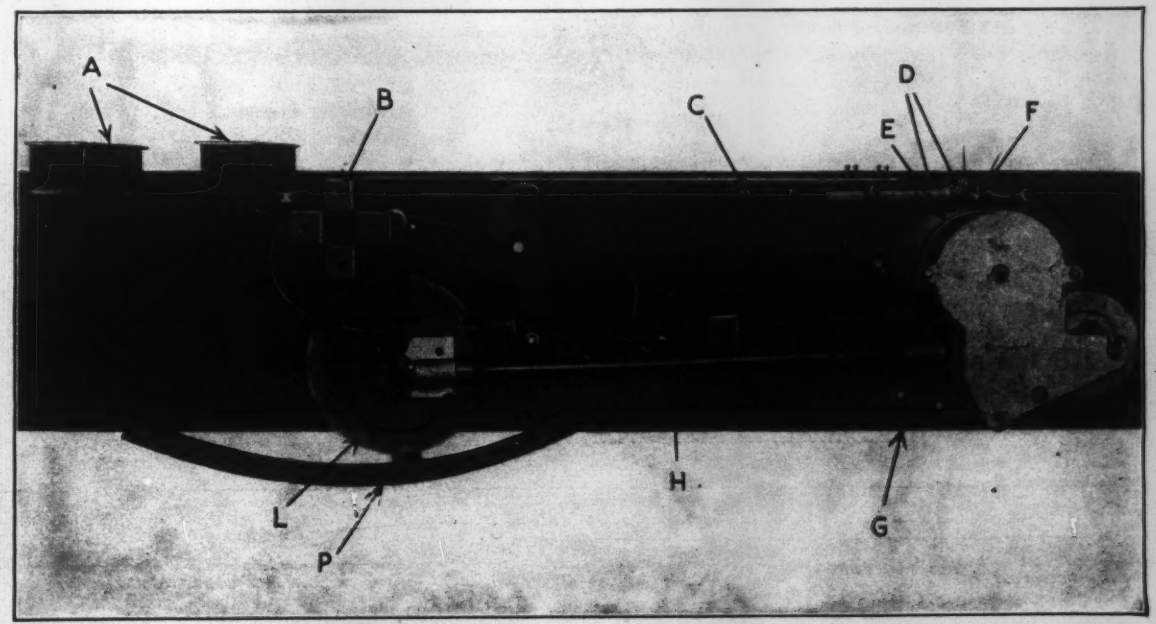


FIG. 2. MECHANISM OF SMOOTHNESS-TESTER, NEAR VIEW; FRONT RUNNER REMOVED

when it descends again the descent registers on the dial. Aftr traveling a mile, the dial therefore indicates a figure representing the total fall in feet per mile, which is called the "coefficient of roughness." Suppose the arc is to 17-in. radius, representing a 34-in. wheel; then the register of 200 for a mile means that an automobile

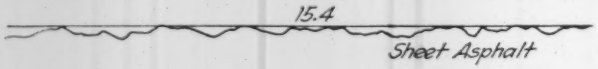


FIG. 3. TYPICAL GRAPH FROM KNEALE MACHINE FOR TESTING SMOOTHNESS OF PAVEMENTS

with 34-in. wheels traveling over the pavement must lift itself vertically 200 ft. in that mile.

Professor Kneale finds that two pavements may give the same coefficient of roughness, but show a very different surface curve. Therefore it is necessary to examine the curve traced on drum *E*, as well as to read the coefficient from the dial.

The vertical scale of the curves traced is 1:1, and as the horizontal scale is 1:96 (1 in. for 8 ft.), a large magnification of the roughness features is obtained.

Track Force for Renewing Rails

In renewing rails in a stretch of railway track the usual practice is to relay one line of rails at a time. There are two methods of handling the work, the new rails in either case being distributed outside of the track. These methods are shown in the accompanying drawing. In the first case, the old rails are shifted outward and the new rails are lifted over them and set in place. In the second case, the old rails are shifted inward to the middle of the track and then disconnected and thrown out after the new rails have been placed. The latter method was advocated in a paper read before the Roadmasters and Maintenance-of-Way Association, by A. M. Clough (New York Central R.R.), who presented diagrams showing the distribution of the gang for the two methods. These diagrams (redrawn) are given in the drawing.

The force for the first method requires 40 laborers, and for the second, 44 laborers. In either case there will be a foreman, two assistant foremen, timekeeper and waterboy. The four extra men required for the second method are employed in throwing out the old rail at the rear of the work. Mr. Clough would add ten men to adze the ties in advance of the rail laying, making a total force of 59 men.

In providing for traffic three or four bolts in a joint are sufficient during the day, but before stopping work for the night all joints should be full-bolted, the bolts made tight and all spikes driven home. To make temporary connections between the old and the new rails it is considered preferable to butt the rails together and splice them with a compromise joint, rather than to close a gap by means of a switch rail butted against one rail and laid against the side of the other.

TRACK FORCE FOR RENEWING RAILS

		Plan No. 1 Rails Thrown Out	Plan No. 2 Rails Thrown In
A.	Flagging	2	2
B.	Taking up and putting down crossing and handling insulated joints	2	2
C.	Handling tools and hand-car	1	1
D.	Pulling spikes	4	4
E.	Throwing old rail out	3	
E.	Throwing old rail in		3
F.	Tong men placing new rails	12	12
G.	Spiking joints and quarters	2	2
H.	Barring new rail into position	2	2
I.	Spiking	4	4
J.	Adzing ties for new joints	1	1
K.	Applying and bolting joints	6	6
L.	Removing expansion shims	1	1
M.	Throwing out old rail		4
N.	Adzing ties ahead of work	10	10
O.	Water-boy	1	1
P.	Timekeeper	1	1
X.	Foreman in general charge	1	1
Y.	Assistant foreman, placing expansion shims	1	1
Z.	Assistant foreman, following up work	1	1
	Total	55	59

To avoid cutting too many rails to make the closure for a splice, the portions of the first rail that is cut should be carried forward and used in connection with short lengths to fill the gap temporarily, until the progress of the work allows a full-length rail to be placed. For convenience the short lengths supplied should have the ends painted green, except that those cut specially for curves should have the ends painted red. The rails should be laid with broken joints, rails 6 to 12 in. shorter than standard being used on the inside of curves to keep the joints on this side from getting more than 6 in. ahead of the middle of the opposite rails.

Care should be taken to see that the new rails are properly supported. Thus if a rail is laid where an old joint was low, the ties should be tamped or blocked up to prevent the rail from being surface-bent by trains passing

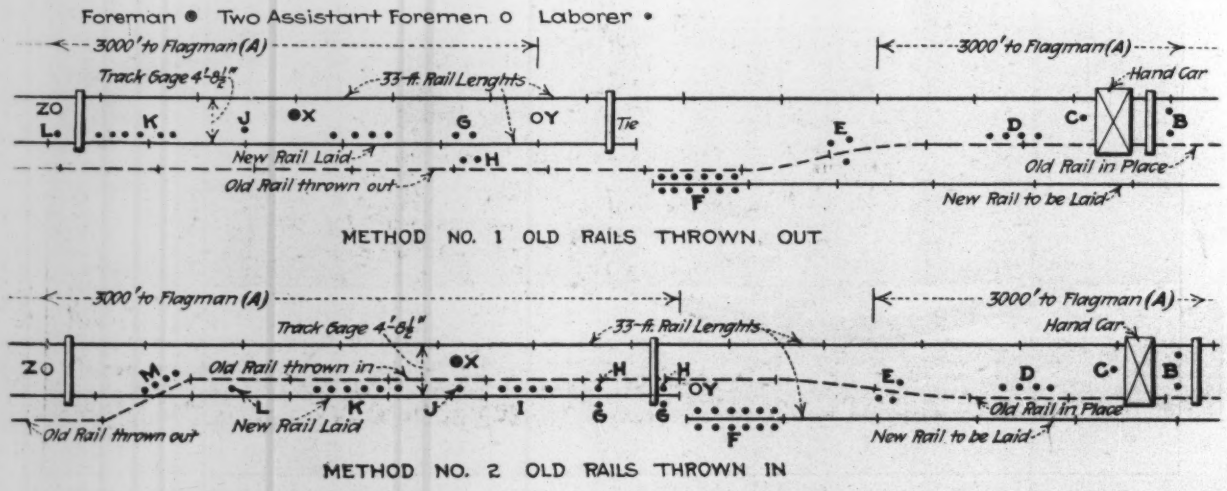

Foreman • Two Assistant Foremen o Laborer •
3000' to Flagman (A)
Track Gage 4'-8½"
33-ft. Rail Lenghts
Tie
Hand Car
New Rail Laid
Old Rail thrown out
Old Rail in Place
New Rail to be Laid
METHOD NO. 1 OLD RAILS THROWN OUT
Old Rail thrown in
Old Rail thrown out
New Rail Laid
Old Rail in Place
New Rail to be Laid
METHOD NO. 2 OLD RAILS THROWN IN

DIAGRAM OF DISTRIBUTION OF TRACK FORCES FOR RENEWING RAILS BY TWO DIFFERENT METHODS

over it. On double track the rail-laying should proceed in the direction of traffic.

J. B. Kelly (Minneapolis, St. Paul & Sault Ste. Marie Ry.) points out, in a paper published in the August *Bulletin* of the association, that in order to make the best progress the distribution of the men must be varied from time to time according to varying requirements and conditions. He sees no advantage in the method of relaying both lines of rail at the same time, either with large or small gangs.

With greater attention to the supply of short rails for closures and for use on curves, there will be a reduction in the amount of rail cut and wasted. For curves the proper number of short rails at each curve should be figured and distributed at the proper place. This was done on a stretch of line 105 mi. long, and only one or two cuts of new rail were made, these being necessary to fit railway crossings. He favors the use of switch points in connecting up the old and the new rails in finishing work for the night. The closures should not be opposite, but should break joint, and a lamp should be hung as a caution signal for trains to pass at slow speed. The night trains should also have orders to run slowly over the new work.

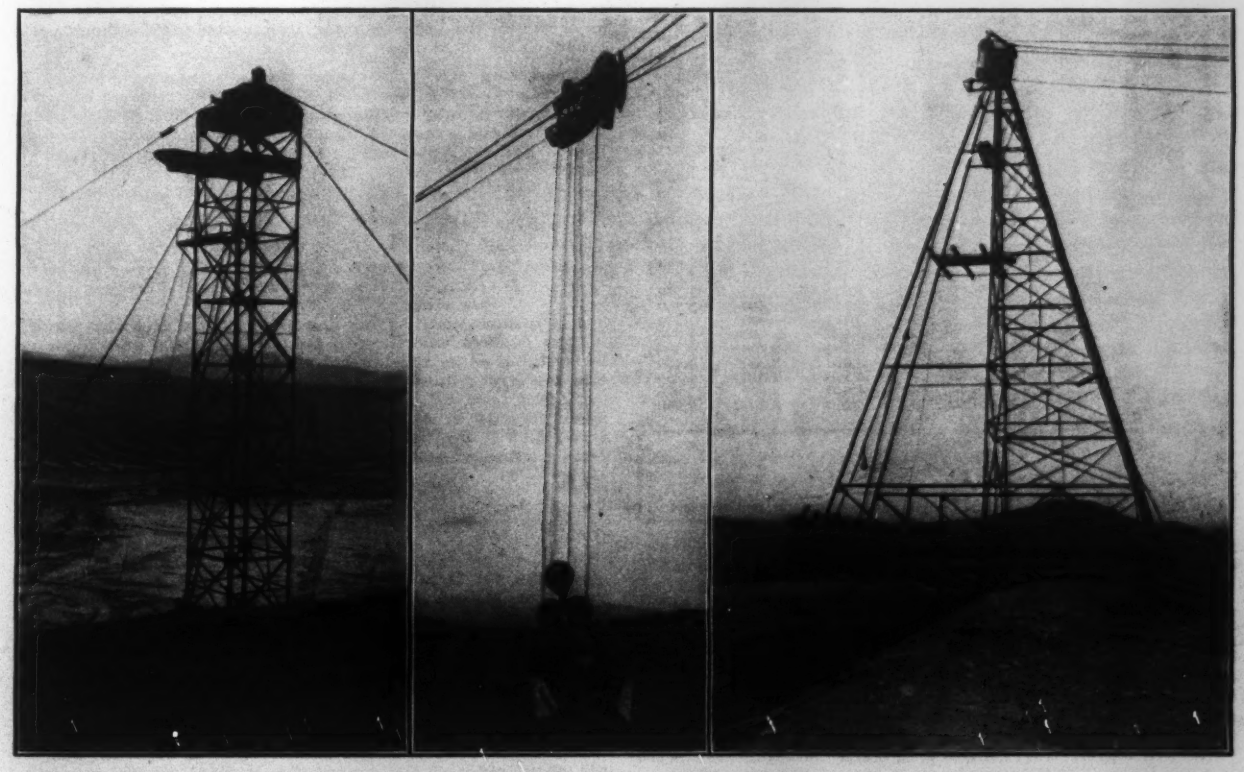
One difficulty, especially with large gangs of 80 to 90 men, is failure to secure proper gage of track without undue work and time. With such a gang Mr. Kelly thinks it well to follow the rail laying with a small crew to finally adjust the track to gage—a matter of great importance both as to safety and as to delay. Before gaging curves the rail joints must be full-bolted and have all the bolts tight or it will be impracticable to keep the track in proper gage.

Temporary Traveling Cableway at Goldfield Mines

A radial traveling, electrically operated cableway of 1,790-ft. span, erected last year, is now in operation at the Goldfield Consolidated Mines Co., Goldfield, Nev., for the transportation of tailings from a waste area to the mill for remilling. The tailings—moist gold-bearing sand that has been worked over once—have been dumped at the present location since 1909 and covers 75 acres. Improvement in the milling process made it worth while to give it a second treatment. The cableway is of the well-known Lidgerwood type and was built by that company.

The radial traveling tower of the cableway is 125 ft. high, and the head tower is 160 ft. high. The tail tower travels through an arc of 109° 40′ about the head, or pivot, tower. Owing to the uneven nature of the ground over which the tail tower travels, it has been found necessary to build a steel trestle in some places to carry the trackway. In the remaining portion the tracks are laid upon the ground. The cableway operates a 4-line, 3-yd. Andresen-Evans bucket, weighing 9,800 lb. The cableway is operated 16 hr. per day in two shifts and delivers 1,000 tons to the mill, which runs continuously to handle this amount.

The grab bucket digs the deposit, conveys and dumps it into a receiving hopper in front of the pivot tower, at a considerable elevation above the tower base. This hopper is mounted to move radially with the cableway travel. From this first receiving hopper a trough on a radial line delivers to the final receiving hopper, from which it passes by the usual method of handling slimes, into a tank and from there it is elevated for retreatment.



CABLEWAY AT GOLDFIELD MINES

Left—Pivotal tower. Center—3-yd. bucket. Right—Radial tower

The three-drum hoist is of the same style as three electric hoists built for the United States Reclamation Service, for use on the Rio Grande project, Elephant Butte, N. M. Each drum is 53 in. in diameter and the motor is of 300 hp., for three-phase 60-cycle current at 440 volts. The main cable is 2¾ in. in diameter.

Safe Distance for Powder Houses

How far powder houses should be kept away from railways, roads and buildings is set forth in a table drawn by a leading authority on the handling of explosives, Col. B. W. Dunn, of the Bureau for the Safe Transportation of Explosives and Other Dangerous Articles. The table is published by the Bureau of Mines in a bulletin[1] just issued.

The recommended distances vary with the quantity of explosive to be stored, up to a maximum of 2,000 to 5,100 ft. (from roads and from buildings respectively) for 200 to 250 tons of explosive.

An important auxiliary rule is given by the authors of the bulletin quoted: "No detonator shall be stored within 100 ft. of other explosives underground, or within 300 ft. of other explosives above ground. No detonator shall be taken into any magazine containing other explosives."

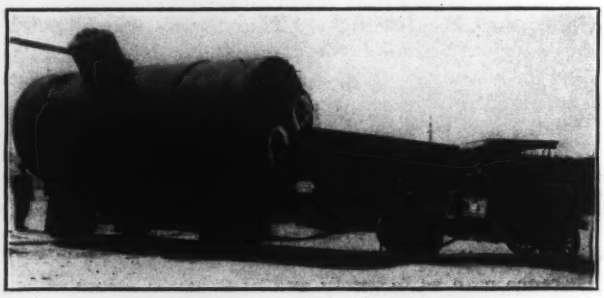
SAFETY DISTANCES FOR EXPLOSIVES STORAGE

Quantity — Blasting Caps, Number	Quantity — Other Explosives, Pounds	Safety Distances, Feet — From Nearest Building	From Nearest Railroad	From Nearest Highway
1,000 to 5,000	.. ..	30	20	15
5,000 to 10,000	.. ..	60	40	30
10,000 to 20,000	.. ..	120	70	50
20,000 to 25,000	0 to 50	200	120	80
25,000 to 50,000	50 to 100	240	140	100
50,000 to 100,000	100 to 200	360	220	140
100,000 to 150,000	200 to 300	520	310	200
150,000 to 200,000	300 to 400	640	380	260
200,000 to 250,000	400 to 500	720	430	290
250,000 to 300,000	500 to 600	800	480	320
300,000 to 350,000	600 to 700	860	520	340
350,000 to 400,000	700 to 800	920	550	360
400,000 to 450,000	800 to 900	980	590	380
450,000 to 500,000	900 to 1,000	1,020	610	400
500,000 to 750,000	1,000 to 1,500	1,060	640	420
750,000 to 1,000,000	1,500 to 2,000	1,200	720	480
1,000,000 to 1,500,000	2,000 to 3,000	1,300	780	520
1,500,000 to 2,000,000	3,000 to 4,000	1,420	850	570
2,000,000 to 2,500,000	4,000 to 5,000	1,500	900	600
.. ..	5,000 to 6,000	1,560	940	620
.. ..	6,000 to 7,000	1,610	970	640
.. ..	7,000 to 8,000	1,660	1,000	660
.. ..	8,000 to 9,000	1,700	1,020	680
.. ..	9,000 to 10,000	1,740	1,040	700
.. ..	10,000 to 20,000	1,780	1,070	710
.. ..	20,000 to 30,000	2,110	1,270	840
.. ..	30,000 to 40,000	2,410	1,450	960
.. ..	40,000 to 50,000	2,680	1,610	1,070
.. ..	50,000 to 60,000	2,920	1,750	1,170
.. ..	60,000 to 70,000	3,130	1,840	1,250
.. ..	70,000 to 80,000	3,310	1,990	1,330
.. ..	80,000 to 90,000	3,460	2,080	1,380
.. ..	90,000 to 100,000	3,580	2,150	1,430
.. ..	100,000 to 200,000	3,670	2,200	1,470
.. ..	200,000 to 300,000	4,190	2,510	1,680
.. ..	300,000 to 400,000	4,670	2,800	1,870
.. ..	400,000 to 500,000	5,110	3,070	2,040

NOTES

Saving $5,000 a Year by Testing hacksaw blades bought by the Public Service Ry., of Newark, N. J., is described by Martin Schreiber, Engineer of Maintenance, in the September "Aera" of the American Electric Railway Association. About 450 gross of blades are used per season in connection with track repair. Ordinarily, prices would be obtained on several makes of blades, and possibly crude tests would be carried out in the field. Finally, the blades would be purchased, taking into consideration the price and the tests. After it was decided to adopt scientific tests, prices were obtained on six types of blades, and samples were bought from the stocks of various dealers. Three blades of each manufacturer were then tested in an Olsen file-testing machine, rigged for the purpose. From the machine, with calculations, the following data were obtained: Total inches sawed, total number of 6-in. strokes, average cut per stroke in inches, total cost of blades per inch cut, total time consumed in minutes, time in minutes per inch cut, total labor cost at 18½c. per hr. (assuming the blades are being used in the field), labor cost per inch cut, cost per inch cut (including cost of blade and cost of labor in using saw), relative cost of each type of blade per inch cut, relative amounts cut by three saws of each type of blade. It was found that blades costing $5.70 per gross showed a total cost, taking into consideration the operation of the blade in the field, of more than 2½ times that of another type of blade costing $5.52 per gross. It was figured by the result of these tests that the company was able to save approximately $5,000 per season on the cost of hacksaw blades.

Single Tractor Hauls Big Boiler—The Henderson Co., a large Philadelphia contracting concern, recently received a contract from the city to move a 33-ton boiler from a power plant in Holmesburg, Penn., to the city power plant at Fairmount Park. It was found that it would take 28 horses to haul this load and 10 extra horses with block and falls to start it. The wagon used to mount the boiler weighed eight tons, making the gross load to be hauled 41 tons. About this time a representative of the Knox Motors Associates made the statement that a Knox tractor, although rated as having from 5 to 15 tons capacity, could easily pull this load. The tractor was brought on the scene and was hitched to the load, as shown in the accompanying illustration. No difficulty was found either in starting or hauling.



TRACTOR HAULING 33-TON BOILER

Correcting River Discharge for a Changing Stage—The following formula has been developed for correcting river discharge for a changing stage. For a description of its derivation and examples of its application in actual use see United States Water-Supply Paper 375-E, by Benjamin E. Jones:

$$\frac{Q_1}{Q_2} = \frac{\sqrt{S_1}}{\sqrt{S_1 + \frac{\text{rate of change of stage}}{\text{surface velocity}}}}$$

Q_1 = Discharge at a given gage height at a constant stage;
Q_2 = Discharge at the same gage height during a changing stage;
S_1 = Slope at the given gage height at a constant stage.

The surface velocity can be obtained from the mean velocity by dividing the latter by 0.85 for small streams and by 0.90 for larger ones.

If K stands for the change of stage per second, V for the mean velocity of the measurement made during the changing stage and N for the coefficient for obtaining mean velocity from surface velocity, the formula may be written

$$\frac{Q_1}{Q_2} = \frac{\sqrt{S_1}}{\sqrt{S_1 + \frac{\frac{K}{V}}{N}}} = \frac{\sqrt{S_1}}{\sqrt{S_1 + \frac{NK}{V}}}$$

Boring for Depth of Railway Fills Over Marshes—In the valuation surveys of the Boston & Maine R.R., now being made, it was suspected that many of the fills made across salt marshes or fresh-water swamps along the coast had settled so that much of the fill was hidden, although no mud-waves or other obvious indications were observed. In order that this important item of cost might not be overlooked, the valuation engineer for the railroad, F. C. Shepherd, M. Am. Soc. C. E., resorted to wash boring. By this means he located the line between the bottom of the sand or gravel of which the fills are composed and the top of the original marsh and found over 2,000,000 cu.yd. of fill, more than was shown by the original cross-sections.

[1] "Rules and Regulations for Metal Mines," by Ingalls, Douglas, Finlay, Channing and Hammond; Bulletin 75.

Editorials

Subway Construction Risks and Reassuring Expert Reports

It is good to learn that experts agree with *Engineering News* in recognizing the existence of an inherent risk in such city work as subway construction and the resulting responsibility of the authorities toward the public. In New York the Mayor's committee of engineers, whose report on subway accidents was summarized last week, laid stress on the inevitable risk, demanded extra safeguards to reduce (not eliminate) the risk and condemned the unstable type of timbering the collapse of which in Seventh Ave. cost eight lives. The experts called from Philadelphia and Boston also mention the risk: "The work is necessarily of a hazardous nature, and some element of risk is inseparable from the carrying on of any such public improvement." They reassure the public by saying that the accidents of three weeks ago cannot repeat themselves. But of course other kinds of accidents may occur, though they do not allude to this possibility.

The city police authorities have taken a common-sense view of the presence of a risk. They have placed certain vehicle-traffic restrictions on the streets in which subway work is being carried on. Though it is a tardy precaution, it is sound in aiming to protect the street-using public. Similarly, the Public Service Commission's orders to the contractors to provide extra bracing are a move in the direction of reducing the risk.

If the city authorities now show an appreciation of their responsibilities to the public as to protecting the street-user from undue risk, the fact only emphasizes their prior neglectfulness in the same matter. It remains true that the authorities permitted, if they did not encourage, the public to consider a timber deck, supported on falsework, over rock blasting and digging, as being just as safe as any normal street. There was a fear of creating a feeling of caution or a tendency to linger less along the timbered streets, because this might affect the business of the property owners along those streets.

Did the fear of creating alarm also control the engineers in drawing their reports on the accidents? Did they consciously make their findings as colorless as possible in the attempt to make them reassuring?

The Seventh Ave. subway collapse of Sept. 22 "was initiated by a blast in the northeast corner of the excavation"—that is the report of the Public Service Commission engineers. From their silence on type of timbering and supervision one must conclude that these were good. The subway experts brought in from Boston and Philadelphia to look over the situation go a little farther and say explicitly that both were good. They say that the timbering was strong; the worst they can say of it is that some details were not in accord with best practice.

Engineering reports that produce results have more life and energy than these and go more directly to the point. The reports just referred to evade the primary question—a question which was placed squarely before the Public Service Commission within a few hours after the Seventh Ave. collapse. The question was not, Why did a collapse occur? A sag or a local collapse due to failure of a support or damage from a blast would have been accepted by even the humblest citizen as a normal mishap. The question was: Why did the collapse extend in such disastrous manner for a length of several hundred feet, far beyond the range of any local accident?

In answer to this very direct and very pressing question the engineers reply that the accident was initiated by a blast in the northeast corner of the excavation. And the chief engineer adds, in further explanation of the rather unusual effect of the blast, that "the blast acted abnormally."

However, longitudinal bracing has been put in the subway timbering at a great many points—in fact, wherever there is a length of more than a few bents of timbering intervening between the excavation space and the permanent subway framework. This measure was due to the praiseworthy promptness of thought and action by the Public Service Commission engineers immediately after the accident. Messrs. Davis and Quimby, the outside experts, seem to think that such bracing is rarely if ever needed. They dwell on the desirability of securely strutting across the subway excavation by continuous transverse struts wedged against the sides, as a means not only of supporting the sides of the cut, but also of making the timbering stable. Longitudinal bracing they admit only grudgingly.

As to the second important question raised by the subway accident in New York—namely, the protection of the sides of rock cuts—the reports leave one quite as much in the air. The Public Service Commission engineers, dealing only with the Seventh Ave. accident, ignore the rock-slide question in their report. The outside experts lay chief stress on it; they say that the rock-slide danger is so great that it is practically the only serious danger menacing the subway construction. Yet finally they come near to retracting all their radical statements, and they do not recommend any measure of all-around protection.

Different passages in their reports, put together, show that they take a conservative and prudent view, but hesitate to express it clearly. They say that the sides of rock cuts should be strutted or shored wherever the rock is loose or subject to loosening. Then they say that strutting and wedging should be used wherever the strata are tilted at an angle so that they might slide or topple into the cut. Again at another point they say that most of the rock which they saw is such that its stability when trenched is not certain in advance, and that in such cases prudence would dictate complete rigid shoring. But when they come to specific recommendations, they advise shoring or strutting only in rock with inclined strata showing open seams.

From the various remarks of the experts one may well draw the conclusion that, were they a little bolder in

their expression, they would recommend as a regular precautionary measure full timbering and bracing of all sides of rock cuts in subway excavation in New York City. Such a recommendation, had they made it, would be a most important one and would inevitably call for very serious consideration.

It is pertinent to note that the Public Service Commission has not issued precautionary instructions concerning the sides of rock cuts either in its specifications or otherwise. Contractors are not very likely to adopt a blanket precaution going beyond ordinary practice until directed to do so. For example, they are not likely to timber an open rock cut unless loose rock is struck. These conditions would have made the recommendation in question a prominent one. In the opinion of *Engineering News* a precaution of this kind would be a most desirable measure of prudence in an extensive enterprise carried on under urban conditions similar to those controlling the New York subway construction work.

Treated Ties Are Little Used

The economy and efficiency of treated timber ties for railway track have been proved in theory and in practice so often and for so long a period of time that it is surprising to realize to what limited an extent these ties have been adopted. In number and in track mileage they make a large aggregate, but the proportion which they represent of the total number of ties and miles of track is comparatively small. Furthermore, this aggregate has been reached only after many years, and the present increase appears to continue at a very slow rate.

This condition is the more surprising when it is considered that in many cases but little increase in first cost is required to obtain the economy due to prolonged life of ties and consequent reduction in the work and expense of track maintenance. This is due, of course, to the fact that the application of a preservative treatment to ties of cheaper kinds and grades (and of inferior quality if used without treatment) may render them equal in quality to high-grade ties of untreated timber.

Estimates of the economy of treated ties are numerous, and one of the latest was submitted at a recent meeting of the operating officers of the Baltimore & Ohio R.R. by F. J. Angier, superintendent of timber preservation of that road. He takes an untreated white-oak tie at 71.7c. and a treated tie at 77.7c. (including 23c. for treatment). With a life of 8 years for the former and 14 years for the latter he arrives at an annual cost per year (including interest) of 24c. and 20c., respectively, or an annual saving of 4c. for each treated tie.

The tie renewals on this road average over 2,000,000 annually, and it is estimated that 1,700,000 of these could be treated with advantage. With a saving of 4c. per tie the net annual saving for the first year would be $68,000, and this amount would be added each year until the whole line is equipped with treated ties. This estimate relates to the ties alone, and Mr. Angier makes no allowance for the resultant saving in cost of track maintenance and tie renewals. But with a large and increasing number of ties of 14 years' life, replacing others of 8 years' life, there should be a material reduction in the track disturbance and the cost of track work due to tie renewals.

In comparison with the 1,700,000 treated ties noted as desirable for annual tie renewals it is found that the numbers of treated ties laid on the Baltimore & Ohio R.R. during the fiscal years 1914 and 1915 were 210,000 and 873,000 respectively. It is probable that few roads can make so good a showing of the proportion of treated ties to the total number of ties used in renewals. Nevertheless, at the end of June, 1915, only 7% of the ties in the track on this road were treated ties.

The foregoing estimate is mentioned in particular not because it is specially better or more important than others but because it is one of the latest, and it serves to call attention again to an opportunity of economy that is ready and waiting for the railways to accept and apply. Similar estimates have always pointed to a great possibility for economy.

Untreated ties continue to form the great majority of the ties used annually for new track construction and maintenance, with treated ties in a decided minority. In view of this condition and of the undoubted and material economies to be obtained by the more general use of the latter, it would be a wise move on the part of railway engineers to bend their energies toward securing a much more rapid and extensive introduction of treated ties.

Latest Canal Slide Formidable

The slide in Gaillard Cut is assuming menacing proportions. In October, 1914, when it first made its appearance, it was estimated to contain about 5,000,000 cu.yd.; in September of this year, when its first serious interference with traffic had been cleaned up, its contents were figured as 7,000,000 cu.yd., after the emergency removal of 2,000,000 cu.yd., and now press dispatches state that there is 10,000,000 cu.yd. to be removed. Apparently the area and volume of this largest of the slides are gradually increasing.

Fortunately the public has become educated up to the slides, so that the latest has not caused the consternation in the press that marked the earlier ones. It is being recognized that the canal has been and for some time will be open only on sufferance and that its commercial and military value can be counted on only after the slipping earth sides of the cuts have finally been leveled down to their required slope.

The main question now is, How long will the present blockade last? This cannot be answered without more information from the Isthmus. If all of the sliding earth were in the channel it would be a year's job to remove it, for in the normally constricted Culebra fairway 1,000,000 yd. a month is almost the limit of excavation. Happily, however, the nature of these slides is such that although ultimately the whole area under motion will have to be removed, the channel section can be cleared up first, always with the proviso that some of the farther earth may at any time slide lower and cause a temporary block. It seems hardly probable, however, that through deep-draft traffic can be resumed before the new year.

While the excavation of the slides is largely a routine matter to the canal engineers, who have been digging now for over eleven years, the added cost to the canal is considerable. Official reports up to now have stated that 30,000,000 yd. of earth has already been taken out from slides at an average cost of from 30c. to 40c. per yd. A total of about $14,000,000 can thus be charged against the insecure geology of the Isthmus.

The Cost of Drainage Pumping

Some very interesting and useful figures on the actual cost of drainage pumping in Louisiana have been collected at great labor and pains by one of the Drainage Engineers of the Department of Agriculture and are printed elsewhere in this issue. The usefulness of the figures is materially increased by the unique tabulation of surrounding conditions. The total operating costs range from 4.1 to 16c. per acre-ft. lifted 1 ft. This means from 3 to 11c. per hp.-hr., with 7c. average, which does not depart from the probable range of figures applying to other small isolated steam plants.

To the annual operating costs, probably some $8 per hp. annual fixed charges on pumping plants is to be added. (The engine and pump equipment for the noncondensing plants would cost some $35 per hp., compared with $45 for the condensing plants, but the boilers and auxiliaries would mount to some $30 instead of $20, so that the total investment per horsepower capacity would remain relatively the same—near $65.) These Louisiana plants operated intermittently from 8 to 145 full 24-hr. days in a year (with an average of 43) so that from 4½c. down to 0.3c. is to be added to the cost per horsepower-hour, the average being some 0.7c. per hp.-hr. This introduced into the tabulated costs in Mr. Okey's article brings the Louisiana figures to from 3.5c. per hp.-hr. at Little Woods to 12.9c. at Poydras—with an average of 8c. for all there shown. For the work of a drainage engineer, Mr. Okey's original figures, expressed for hydraulic units, will be the more useful. For the man approaching drainage from the power viewpoint, the comparisons given above may be instructive—at least they will show that general knowledge of power costs on a horsepower basis can be safely applied to the preliminary study of drainage and irrigation problems.

⁂

The example set by the Engineers Club of Kansas City, in securing the services as a lecturer on military engineering of Major S. A. Cheney, U. S. A., is well worthy of emulation by similarly constituted societies. There are few such societies that are not located at or near an army engineer station, where may be found an officer who doubtless would consider it almost a part of his duty to assist the civilian engineer to a better understanding of the details of military engineering. Furthermore, the state of the public mind makes this the one time to insure a good response on the part of the civilian engineer.

⁂

Los Angeles, Calif., which is noted for its good street surfaces, had 792 automobile accidents during August. Good pavements and a high accident record may be related, as the temptation to speed varies directly with the excellence of the highway. Such an accident record suggests that traffic regulation is not what it should be in Los Angeles. When a city improves its highways it is apparently desirable to brace up its traffic squad at the same time.

⁂

Do local sections of national engineering societies tend to disintegrate the older type of local societies without commensurate benefit to the members of the local sections? A number of engineers in one city say yes. What do others with experience think—in thoughts of say 25 to 50 short words?

Letters to the Editor

Patentees and Royalties

Sir—Some rather surprising mathematics is produced by R. C. Hardman, under the title of "Patentees and Royalties," in his letter in your issue of Aug. 26, 1915. He attempts to show that the patentee has an advantage over his competitors, when all are bidding on the patented plans, the royalty payable to the patentee by his competitors being publicly specified. I had contended in your issue of July 29, 1915, that the patentee has no advantage over the other bidders.

As an illustration in support of my view, let us suppose that Contractor Smith ships $1,000 in gold by boat. The vessel sinks and the owner calls for bids to raise the ship, specifying that the successful bidder shall recover Smith's $1,000 and restore it to him. Contractor Smith concludes to bid himself. How much advantage has he over his competitors? None, of course.

Suppose that instead of a chest of gold buried in the sea, Smith owns a patent on the method of raising the vessel and the owner specifies that Smith should receive $1,000 out of the salvage for his royalty. What advantage has Smith over his competitors, if he concludes to bid himself? None, of course.

The answer to Mr. Hardman's mathematics is that it was the profit that was reduced in each case, and not the royalty. Mr. Hardman assumes that "the work presents few difficulties so that the profit is fairly assured." But the royalty is *absolutely* assured. Why, then, risk any of it? If the patentee-contractor takes the work, it can only be because he has reduced his profits below those of his competitors, and not the specified royalty.

Mr. Hardman intimates that it makes no difference anyway, whether a reduction is made on profit or on royalty. But the difference is just this, that if the reduction is made on royalty it takes money out of the bank, whereas if it is made on the profit it merely reduces the odds on a speculative item.

Let us examine the problem in another way. If the specified royalty gives the patentee an advantage, that advantage will be increased by making the royalty excessive. Let us assume, then, that the work is to cost $1,000 and that the royalty is to be another $1,000 and that to this total cost of $2,000 we add no profit. Now let the patentee bid against his competitors. How much can he cut his bid to get the work? According to Mr. Hardman, he will sacrifice any part of the $1,000 royalty, but if he reduces the royalty by even one dollar, he will

lose money because he will then realize but $999 while the successful bidder would pay him $1,000. Suppose we figure a profit of $1 instead of nothing. Then the patentee-contractor can reduce his bid by any part of that one dollar to secure the award. But when he deducts more than one dollar he loses money. The same reasoning applies as the profit is increased and the royalty reduced to reasonable amounts.

As to Mr. Hardman's actual case, where a cut of $300 was made by the patentee-contractor to permit award at private letting, the answer is the same—that the profit was reduced and not the royalty. But Mr. Hardman even says that it matters not which was reduced. Surely, he has no complaint if his competitor was willing to accept less profit than himself. There is nothing dishonorable in presenting a supplemental bid. Mr. Hardman admits that the statute was complied with. In Missouri the law encourages supplemental bids, and he is evidently from that state.

When the patentee-contractor himself is a bidder, he is actually at a disadvantage, since it is to his interest to supply as nearly perfect a product as possible, whereas his competitors will endeavor to construct at lowest possible cost.

The foregoing illustrations will be found to apply regardless of the merit of the patent or the amount of the royalty, and equally when a patentee or his agent on commission is one of the competitors. The principle may be stated as follows: A patentee-contractor who is entitled to a specific payment from the successful bidder must add that amount to his computed cost if he elects to lose that payment by defeating his competitor. Every other bidder adds it to his cost. Hence all are on an equality.

In this principle, too, will be found an excellent test of breadth of reasoning. Engineers, who are accustomed to view but one side of a case, will find it hard to accept the principle even when demonstrated. Lawyers, who are trained to view both sides with impartiality, will accept it, and the courts hold in accordance with it. A bookkeeper or accountant will grasp the principle readily, but the ability to do so seems to be strangely lacking in the reasoning powers of many of the bidders that compete on patented devices at public lettings.

DANIEL B. LUTEN.

Indianapolis, Ind., Sept. 9, 1915.

Gulf Storms and the Recent Galveston Flood

Sir—I have had to do in various professional ways with eleven points along the coast of the Gulf of Mexico from Gulfport on Mississippi Sound to Coatzacoalcos on the Isthmus of Tehuantepec. I have lived through the storms, the hurricanes and the "northers"—have seen their coming, their activities and their effects. The history of my works there goes back to 1875.

I have read the descriptions of the Galveston disaster in your issues of Aug. 26 and Sept. 2. I have looked for a very important physical feature, but see no special reference to it. I refer to the fact that the gulf storms usually start in the southeast. The wind blows from that quarter for a day or two, steadily growing stronger and stronger. This storm, originating in the central portion of the gulf or even at times in the Caribbean Sea, piles the water up along the northern shore of the gulf, often so high as to be called a "tidal wave." The storm develops into a wind of hurricane velocity before it changes. It then moves around in the direction of the hands of a watch, into the south, southwest, west, and finally quickly whips into the northwest or north, when it blows sometimes with terrific force—75 to 100 miles an hour.

At most points—the mouth of the Mississippi River, Sabine Pass, Indianola (wiped out by the southeast gale in 1875) and elsewhere—the north wind brings relief from the high water, but at Galveston the situation is different. The southeast winds force the rising waters through the two channels, east and west of Galveston Island, up into Galveston Bay, which has an area of 271 sq.mi. When the wind veers into the north the immense mass of water piled up in this great bay is forced toward the gulf. The channel cannot immediately carry it, and it rushes over the island itself, but it is obstructed by the high water which is already on or around the island and outside in the gulf and which, under the reserve force of the southeast hurricane, is still striving to get into the bay, and these two masses of water moving in opposite directions, or striving to do so, pile up on Galveston Island lying between the bay and the gulf.

There is only one remedy. Do as has been done at Cairo, Ill., at the confluence of the Ohio and Mississippi—raise the entire city or at least surround it by walls high enough to keep out the highest floods and tides.

Galveston will have disasters similar to this last one, and perhaps worse, until the entire city is raised at least 16 ft. on the gulf side and 10 ft. on the bay side.

The remark in your editorial of the Aug. 26 issue is correct: "The fact that most of the flood damage seems to have been on the north side of the island, where grades are still as low as they were before 1900, *points the way to future work.*" The italics are mine. I might add that in 1900, in the American Church at Buenos Aires, before a large and interested audience, I delivered an address just after the disaster of that year, described the geographical, physical and meteorological hydraulic situation as here briefly given and predicted the disaster of this year—15 years later.

E. L. CORTHELL.

North Egremont, Mass., Sept. 4, 1915.

Proper Timbering Would Have Prevented Subway Collapse

Sir—In reference to your articles on the subway timbering accidents on Seventh Ave. and on Broadway and the editorial on the same subject, Sept. 30, I would like to offer some comments.

In these articles the system of timbering in use at the scene of both accidents is described in detail with illustrations, as well as the conditions after the accidents. It seems to me that the causes of both accidents are entirely apparent from these descriptions and I do not therefore agree with the vague and indefinite suggestions mentioned as possible causes.

To quote from *Engineering News* as to the timbering in Seventh Ave:

The beams under the street planking and car tracks are carried by one 12x16-in. longitudinal stringer on either side of a center aisle. Rests of 6 to 8 posts, spaced 10 ft. apart along the work, support the stringers and also directly the roadway beams. These posts in turn foot on transverse 24-in. 80-lb. I-beams, one 25-ft. length on either side, set 12 to 15 ft.

above the floor of the excavation. Four 12x12-in. posts, two under each I-beam, carry the load of the entire bent. For bracing, the I-beams are strutted longitudinally by 4x8-in. or 8x8-in. spreaders wedged between their webs. Two or three such spreaders are used on either side of the central aisle. Also, the upper tier of posts is set on 4x12-in. or 6x12-in. foot planks dapped over the top flanges of the I-beams in alternate bays between bents. Longitudinal spacer planks under the roadway beams also act as struts. Transversely, the two I-beams of each bent are spread and tied by a pair of 12-in. 20½-lb. channels spanning the middle aisle and bolted to the I-beam webs. There are other spreaders, and at many places transverse diagonal bracing in the upper tier of posts, as indicated in Fig. 5. The outer posts are blocked against the side walls of the cut. There is no longitudinal bracing in either upper or lower tier of posts.

From the above description it is clear that the transverse 80-lb. I-beams in each bent (two of which, together with the center 12-in. channels, were bolted together to form a continuous transverse member) inserted in the line of posting broke the posts at this line, toggle jointing each bent, and that there was no longitudinal vertical diagonal bracing of the posts above or below these cross-members. Further it is evident that the toggle joints of the bents were all locked or joined together by the longitudinal struts between the transverse I-beam members, so that a longitudinal end thrust, applied in line of the struts, would break or fold all the bents simultaneously and allow the street structure to collapse. It seems only common sense to assume that this is what actually occurred in Seventh Ave., the blast supplying the end thrust. When the toggle joints of the bents were broken by a south thrust, the result was that the post props were knocked out from under the temporary street platform and it fell into the ditch.

The accident could have been prevented by longitudinal vertical diagonal bracing of the posts above and below the transverse members. Such bracing would have confined the effect of the blast that directly caused the accident to a small injury of local timber only.

As to the Broadway accident, the timbering conditions were quite similar to those in Seventh Ave., and the extensive collapse of timber structure caused by the rock slide is attributable to the same lack of diagonal bracing. The rock slide itself could have been prevented by proper timbering, similar to that employed in subway excavating in earth; that is, cross-bracing and rangers and vertical lagging blocked against the rock face.

In view of the foregoing, therefore, various explanations of the accidents that have been offered seem unnecessarily vague. I do not agree with some of the reasons mentioned in *Engineering News* of Nov. 30. To quote from the editorial, in which it speaks of the coincidence of the two accidents:

> To conclude anything from the fact of the coincidence is wrong in the present case, however, because the two accidents were fundamentally different in character. The Broadway accident was due to what the lawbooks call an act of God, in this case a catastrophal rock fall. Rock falls are not quite so unforeseeable as hurricanes and floods, but they remain the ever-continuing threat in rock excavation, whether for mine or structural use. Mining engineers, who boast of the perfection of their timbering practice, admit that the heaviest proportion of mine fatalities is due to rock falls.
>
> The Seventh Ave. collapse is unquestionably a very serious matter. But any thoughts or sentiments based on it should not be complicated by reference to the Broadway accident.

Why lay the Broadway accident to "an act of God," when a little timbering, properly placed, would have held the rock safe. Also, as to the statement that "mining engineers, who boast of the perfection of their timbering, admit that the heaviest proportion of mine fatalities is due to rock falls," there is no close analogy between the Broadway accident and the dangerous conditions met with in underground mining. In mining, heavy pressures on the roof and the sides of stopes or other openings are often developed that no timber will hold; also most rock falls underground are from the roof of the opening or stope, where the continuous mining of ore prevents secure timbering of the roof at the point of attack. The Broadway subway cut, however, is from a mining standpoint a shallow open cut the walls of which could be easily held by proper timbering, no matter how the dip of the strata or cleavage planes.

There is however a close analogy between the Broadway and Seventh Ave. accidents, in that the failure of the timber structures was communicated over a much greater area than that directly affected by the rock fall and blast. Diagonal bracing of the posts on Broadway would have restricted the area of timbering affected by the rock fall, and it seems to be only a matter of luck that a greater area did not collapse. This diagonal bracing has since been put in on the Broadway section in question and elsewhere.

JOHN SEWARD.

West Nutley, N. J., Oct. 6, 1915.

Percentage-Basis Contracts with Economy Clauses

Sir—I notice in *Engineering News* of Sept. 2 an editorial concerning an inquiry as to contracts made on a percentage basis in which there would be an inducement for economy.

A contract of this character, where the writer was engineer, was made by the Troy Public Works Co., of Utica, N. Y., with a private owner for the construction of a macadam road about 2 mi. in length. The contractor was to furnish all materials and labor, also all incidental expenses at actual cost plus 15%. If the cost exceeded the amount as estimated by the engineer according to the stated schedule of quantities and prices, the contractor was to receive only 7½% on such excess. If less, the contractor would receive, in addition to the 15% of actual cost, 10% on the difference between the cost and the original estimate. However, this contract was not completed, as after the work was well under way construction was discontinued by order of the engineers of the New York Water Supply (since the road went over a hill which will be surrounded with water in the new Kensico reservoir) and settlement was made for work done by mutual agreement. Later a like contract was made between the same parties for steam-shovel work at another place.

E. F. MUSSON.

Norwich, N. Y., Sept. 8, 1915.

A Water Storage Dam 140 ft. high, impounding 3,500,000,000 gal. of water for regulating the flow of the Naugatuck River, Connecticut, has been recommended by Charles H. Preston, Jr., of Waterbury, Conn., in a report to the manufacturers of the Naugatuck Valley. The plans for the dam have been accepted by the State Board of Engineers. The dam, which is the first of a proposed series, would contain 190,000 cu.yd. of cyclopean masonry. The entire project, if carried out, would be the largest enterprise of the kind in New England, and would cost several million dollars. The prospect for its construction is not announced.

New Water-Conservation Scheme at Fall River, Mass.*

SYNOPSIS—Four conduits in a single three-story reinforced-concrete structure would convey pure lake water for power, divert sewage, turn surface water at will into a pond for storage or into a river to waste, and convey condenser water into oil-separating and condenser basins. Water-power for mills would be regulated, the city water-supply increased, river-pollution nuisance abated and 146 acres of land reclaimed.

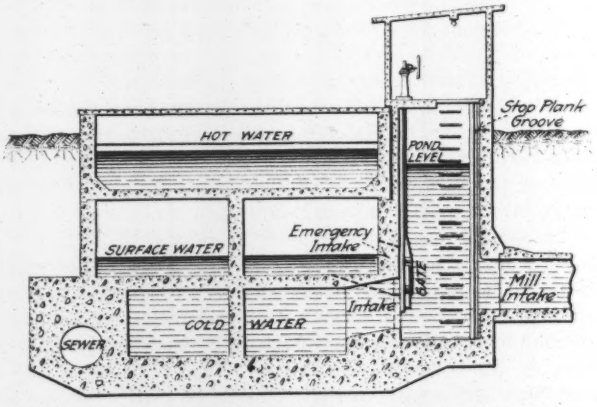
The city of Fall River, the principal cotton-manufacturing center in the United States, is crossed transversely by the Quequechan River, a stream having a total length of only 2½ mi. The stream serves as an outlet for the North and South Watuppa ponds, which together constitute in area the largest body of fresh water in Massachusetts, and which are only 2 mi. from tidewater and 130 ft. above it. In its first 2 mi. the river's course is nearly level, the 130-ft. drop taking place in a series of falls occurring in the last half mile of its length. The Watuppa Dam, built many years ago at the point where the river begins to fall, has caused the upper portion to overflow the flats adjoining the original river bed, so that at some points the river is 2,000 ft. wide.

Upon the shores of the Quequechan are located numerous cotton mills containing about one-half of the 4,000,000 spindles in the city. These mills draw water from the stream for condenser purposes and for bleaching and washing fabrics and return into it the heated water which has been passed through the condensers. They also discharge into it various wastes, including sewage, and the stream is further polluted by oil from the condensers, by surface water from the slopes of the city adjoining the stream and by city sewage discharged from the sewers through various overflows. The larger number of these mills are located along the upper, overflowed reach of the river, which by reason of the pollution which it receives is at times very offensive.

Of the two Watuppa ponds, the North Pond is utilized as the source of the city's water-supply, and is in fact scarcely sufficient for that purpose, and must soon be augmented from another source. This leaves only the South Pond available for supplying through the river the demands of the mills along the stream. This supply varies greatly in quantity with the level of the pond and at times is insufficient to furnish the mills with the desired quantity of water, while at other times much is wasted. Moreover, the hot condenser water discharged from the mills nearest the lake heats the river water to such a degree that mills farther downstream frequently find it necessary to shut down temporarily, as the water is too hot for condensing purposes. At certain times the water is too polluted to serve for washing fabrics.

The improvement of the stream and conservation of the water have been under consideration for about 40 yr., but only within the last year has a complete study, accompanied by plans and details of cost, been made.

The most interesting and unusual feature of the project proposed by the commission [created by the legislature of 1913] for the solution of the problem is the main conduit, a three-level reinforced-concrete structure of large size, which is here shown in cross-section. This structure extends from the South Pond to the Watuppa Dam and has a total length of 9,800 ft. In its lower portion it contains a covered channel sufficiently large to take the entire river discharge and at a sufficiently low grade to enable the lake to be drawn down considerably lower than is possible through the natural river bed, thereby increasing by about 70% the available storage capacity of the lake and insuring for the mills an abundant and regular supply of water of the same purity and temperature as the lake



PROPOSED MULTIPLE WATER AND SEWAGE CONDUIT

water. This channel is level throughout its course and is under pressure; in effect, it extends the lake to the mills. Immediately above this channel is one providing ample capacity for storm water. This channel is also level, and is fitted with controlling devices that permit the storm water to be discharged either downstream into the Taunton River or upstream into the South Pond. At the uppermost level is a covered channel to take the heated and polluted condenser water back to the pond, into which it is discharged after passing through both an oil-separating basin and a cooling basin. In the foundation of this structure, adjoining the cool-water channel, a trunk sewer is constructed for a portion of the length of the conduit. The areas and capacities of the various channels in the portion of the main conduit adjoining the pond are as follows:

	Area, Sq.Ft.	Capacity, Sec.-Ft.
Cold-water channel	110	500
Surface-water channel	143	700
Hot-water channel	81	225

The overall dimensions of the conduit at this point are 37 ft. in width and 23 ft. in height. The cold water is discharged downstream into the existing river bed at a point where the river begins its rapid descent. The sewage and storm water, however, are discharged into an outfall sewer, built as a tunnel and emptying into the

*Abstract of report of the Watuppa Ponds and Quequechan River Commission to the City Council of Fall River, Mass., Fay, Spofford & Thorndike, Consulting Engineers, 308 Boylston St., Boston.

Taunton River. As a protection in case of a severe flood, this tunnel is made somewhat larger than is necessary for sewage alone, and is also connected with the cold-water channel, thereby making possible a flood discharge greater than can now be conveyed by the existing river channel between the Watuppa Dam and tidewater. This portion of the river bed is badly obstructed by various deposits and is contracted by the construction of buildings alongside and over it. Suitable provision for longitudinal expansion is made in the conduit, and special precautions are taken to allow for the independent movement of the hot-water channel.

One of the chief advantages of the conduit is the opportunity it provides for increased fire protection for the mills along the river. The abundant supply of water in the cold-water channel can be readily utilized for fire protection, not only for the district in the immediate vicinity of the conduit, but also in the portion of the city lying along the river valley. The district to which this additional fire protection can be secured includes the central business section of the city, in addition to the manufacturing district bordering the river. The advantage of this additional fire protection is particularly important because of the limitations of the present domestic water supply.

Another important advantage of the plan, so far as the water-supply is concerned, is that by its capacity of lowering the lake to considerably lower levels than can be done through existing means, a far greater storage capacity of the lake is made available and a regular daily supply of 20,000,000 gal. throughout the year can be obtained, this being equivalent to the average yield of the watershed. Under the conditions now existing the daily water-supply may fall as low as 5,000,000 gal.

In addition to the design of the main conduit numerous branch conduits to supply the various mills have been planned, as well as a complete system of sewers for the unsewered portion of the city adjoining the stream, and storm drains where needed. Only a limited portion of these sewers and drains need be built at the outset.

In the process of the investigation considerable attention was given to the separation of oil from the condenser water to prevent its being discharged into the pond, thereby forming a nuisance because of its deposits along the shore of the pond. Experimental studies were conducted on this topic in the laboratories of the Massachusetts Institute of Technology, which resulted in establishing the size and type of the oil-separating basin. Similar study was given to the question of cooling this hot water to prevent undue heating of the lake water near the cold-water intake.

It was found that a cooling basin sufficient in size to permit of cooling the discharged water in very hot weather to the same temperature as the lake was impracticable. The cooling basin as designed will, however, have considerable effect in this direction, and its outlet into the lake is sufficiently remote from the cold-water intake to insure that the partly cooled water will not reach the intake before thorough mingling with the lake water; it will therefore have sufficient additional cooling to make sure that the water reaching the mills will be cool enough for condenser purposes under any conditions that may arise.

The plan also contemplates as a necessary part of the abolition of the nuisance the filling of the entire area of the flats. This will result in the redemption of 146 acres of land, which will immediately become available for manufacturing and civic purposes.

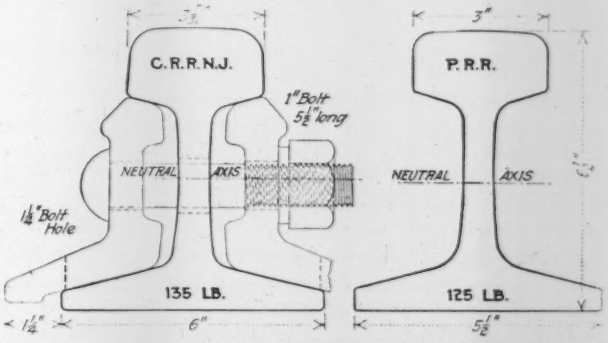
The total estimated cost of construction is $2,592,500, and in addition to this the engineers have made an allowance of $100,000 for damages and $250,000 for general expenses, bringing the total gross cost up to about $3,000,000. As an offset to this it is estimated that the value of the reclaimed land will be $800,000 and the benefits accruing to the mills $1,000,000, thus reducing the cost to the city to somewhat over $1,000,000.

According to the legislative act, the project must be adopted or rejected by the city council within thirty days of the receipt of the report. The project has already been approved by the Massachusetts State Department of Health.

Two Heavy Rail Sections

Rails heavier than 100 lb. per yd. are exceptional, and have been applied mainly where conditions of curvature and heavy traffic combine to cause severe stress and wear upon the track. The Pennsylvania R.R., however, is introducing a 125-lb. rail for general use on its busy main lines, and the New York Central R.R. is using a 105-lb. rail for similar lines.

The heaviest rail now in use is the 135-lb. rail of the Central R.R. of New Jersey. This was adopted a few years ago and is now in service on about 40 mi. of track. The Pennsylvania R.R. has more recently adopted its 125-lb. rail. Last year 15,000 tons of this section were laid, covering about 75 mi., and the 1915 contracts call for 37,500 tons.



DESIGN OF RECENT HEAVY RAIL SECTIONS

	C.R.R. of N.J.	Penna. R.R.
Weight per yd.	135 lb.	125 lb.
Height	6½ in.	6½ in.
Width of base	6 in.	5½ in.
Width of head	3 5/32 in.	3 in.
Sides of head	Sloped	Vert.
Depth of head	2 in.	1½ in.
Depth of web	3 9/32 in.	3 13/32 in.
Depth of base	1 7/32 in.	1 7/32 in.
Thickness of web	½-in.	21/32 in.
Radius of		
Head	14 in.	12 in.
Top corners	½ in.	7/16 in.
Bottom corners	3/16 in.	1/16 in.
Sides of web	14 in.	16 in.
Top fillets	½ in.	½ in.
Bottom fillets	½ in.	½ in.
Corners of base	½ in.	1/16 in.
Head	40.28%	38.9% (4.73 sq.in.)
Web	21.90%	20.3% (2.47 sq.in.)
Base	37.82%	40.8% (4.95 sq.in.)
Moment of inertia	72.39	68.7
Fishing angle, top	14°	18°
Fishing angle, base	14°	14°

Sections of the two rails and their comparative dimensions and proportions are given herewith. It will be seen that they represent distinct types of design.

Public-Service Engineers on New York Subway Accidents

SYNOPSIS—The engineers of the Public Service Commission of New York City report that the Seventh Ave. collapse of Sept. 22 was caused by a blast in the northeast corner of the excavation. E. S. Davis and H. H. Quimby report that the subway timbering has only minor defects, that continuous longitudinal bracing is unnecessary and that the only danger is from rock slides.

Reports have been made on the two recent failures of subway timbering in New York City (Sept. 22 on Seventh Ave. and Sept. 25 on Broadway) by the engineers of the Public Service Commission, which is in charge of building the subways, and by an expert board composed of Edmund S. Davis, Chief Engineer, Boston Transit Commission, and Henry H. Quimby, Chief Engineer, Philadelphia Department of Transit. The latter report deals with both collapses, but the engineers of the Public Service Commission say nothing about the Broadway accident and confine themselves to the earlier one. The chief points in the reports are as follows:

Cause of Seventh Ave. Collapse

John H. Myers, Division Engineer of the Public Service Commission, whose division included the Seventh Ave. section where the collapse occurred, reports that just before 8 a.m., Sept. 22, the bench on the east side of the north heading at 25th St. and Seventh Ave. was being blasted. One hole near the bottom was shot and then four holes in the bench. The collapse followed immediately after the second blast. The rock, while quite hard, appeared to have a number of seams, and marks at the holes indicated that the rock probably broke out more readily than normal, and may have been thrown a considerable distance back from its original position. Large pieces of rock were found near the wreckage of the tower supporting the middle runner beam (see sketch of timbering, Fig. 5, *Engineering News*, Sept. 30, p. 664). They were probably some of the pieces blown out by the blast.

The eastern one of the three runner beams had been forced south, carrying with it two transverse I-beams (20 in.) suspended from it. Most of the easterly transverse I-beams which divided the street timbering into two stories were found lying in a general southeast-northwest direction. Mr. Myers states:

> I formed the opinion that the blast had been the cause of the accident and that its force had caused the members forming the support of the street and trolley tracks to move to the south, and that this southerly movement was especially evident on the east side of the cut, which was the side on which the blast was fired. I also felt that this movement had in all probability been augmented by the fact that the trolley track, having had the supports taken away from beneath it, had hung in a manner similar to a rope, and that this action by either pulling the posts which were above the cross I-beams to the north or else pushing them to the south had in turn augmented the movement which I believe the blast initiated, and when this disturbance of the track reached the point where the surface car was, its weight added to that of the track increased the force tending to move the supporting structure southward. The car being a northbound one was on the east track, upon which side of the cut there is most evidence of this movement. The posts supporting the cross I-beams fell with tops to the south and bottom ends to the north.

Marks on the web of the most northerly transverse I-beam appear to show the point where a flying piece of rock hit this beam and, according to the explanation given by Mr. Myers, started the collapse.

Robert Ridgway, Engineer of Subway Construction, reports: "My study of the matter has led me to the conclusion reached by Mr. Myers, that the initial cause of this accident was a blast which was fired in the northeast corner of the excavation."

He further states that immediately after the accident he instructed the division engineers throughout the subway work to have a thorough inspection of all shoring and street supports of the subway excavation made at once and take steps to remedy any unsatisfactory conditions, all of which instructions were promptly followed.

Chief Engineer Craven's Opinion

Alfred Craven, Chief Engineer, says: "I am of the opinion that the blast which immediately preceded it, and which I believe acted abnormally, was the direct cause of the collapse of the street decking." However, he also considers that the street-railway track structure played a part in furthering the collapse. This structure (composed of yokes, conductor-rail conduit, conductor rails, track rails and concrete encasement) was very heavy and had considerable tensile strength available for suspension-rope action. When it tore apart, after taking a considerable sag, it exerted an unusual strain on the timbering. Concerning the strength developed by this structure, Mr. Craven says: "The track conditions of the underground trolley exhibited unusual strength under longitudinal strain." And he says that had it not been for the track he believes the timbering "would have withstood the shock of the blast and would not have collapsed."

Mr. Craven reports that the tower-and-runner-beam method of supporting the roof at the headings is generally in vogue and has been deemed safe and adequate. Since the accident showed further safeguards to be necessary, steps to provide them were taken immediately. Swaybracing has been put in place near the headings.

Davis and Quimby Against Longitudinal Bracing

Henry H. Quimby and Edmund S. Davis report that there is little to criticize in the timbering used in the subway work. Quoting from their report:

> The types of timbering used by the different contractors vary somewhat. With special reference to the method used by the United States Realty and Improvement Co., in whose work the two recent casualties occurred, we found that the sizes of the material used for the various members give ample strength for the loads imposed by the street traffic, and we are of the opinion that while some cases of details of the framing and bracing were found that are not in accord with the best practice, the greatest and only serious menace to the shoring and to public safety consists in the possibility of rock slides. . . .
>
> The cases wherein details of the timbering have appeared to us as unsatisfactory are the omission of efficient lateral stays at some joints in the posting, points where needles or cross-beams rest upon posts and carry upper-story posts without struts to stay the points against lateral movement, or where no lateral stiffness has been given by the use of fish-plates or cleats to stay the points against buckling, some

cantilever beams carrying upper-story work without struts at buckling points, and the omission of posts under rangers.

The omission of longitudinal struts at or near the posts of the timbering is not general, the instances of the omission being probably oversights on the part of the workmen, or possibly due to removal either by intention or by accident. Such points in the construction should be the subject of repeated inspection, and the whole structure should be continually maintained in a state of completeness and tightness.

Longitudinal diagonal bracing is probably never needed except in the event of the accidental destruction of one or more of the bents, and as it is necessary to remove it before the permanent steel framework of the subway can be erected, it would seem that a continuous system of longitudinal diagonal bracing is scarcely justified. If a considerable stretch with many bents must be maintained between the permanent subway construction and the heading, the distance might be divided into braced portions to be determined according to the local conditions—say by using diagonal braces at intervals of 30 to 50 ft.—and such diagonal braces might better consist of heavy timbers extending from the needle beams to the floor, so detailed that they can easily be shifted as the permanent construction is advanced.

The lesson of the extensive collapse in Seventh Ave., where the blast that caused the disaster perhaps directly destroyed only the nearest bents, would logically be that, as provision against the occurrence of an accident of any sort that may destroy one portion of the timbering, some diagonal bracing should be provided at such intervals as may be justified by the local conditions. If the permanent subway construction should be kept as close to the heading as is practicable and prudent, considering its liability to injury by blasts, manifestly very little of such longitudinal diagonal bracing will be needed.

Rock Slides Are the Chief Danger

Concerning rock slides, however, these experts are very apprehensive. Under conditions such as prevailed in the Broadway excavation at 38th St., where the second collapse occurred, they think it is highly important that adequate supports be provided for all loose rock by putting in rakers or horizontal shoring or by doweling into the solid rock, wherever the condition of the rock indicates liability of a stratum to separation from its bed. They say:

Much of the rock in the excavations is of a nature that makes it subject to loosening in the seams when exposed to weather, temperature changes and vibrations, and the determination of liability to slide is often difficult. At such points, if any doubt exists as to the necessity for shoring, the doubt should be resolved on the side of safety and no chance should be taken. . . . An efficient system of supporting, strutting and wedging should be at once adopted and installed throughout every area in which the strata are tilted at an angle that will subject them to either sliding or toppling.

The character of most of the rock that came under our observation in the excavations is such that its stability when trenched cannot be certainly determined; and especially when its exposure to weather and to the vibration of street traffic is to be continued over considerable time, as is said to have been the case in the Broadway cut at 38th St., prudence would dictate that complete rigid shoring be provided to secure it.

With regard to both the danger of disintegration of rock and the danger of loosening of the timber falsework, these experts recommend that the permanent subway construction should follow closely upon the excavation, so that as little falsework as possible should stand between heading and permanent structure.

Recommendations of Davis and Quimby

They recommend four specific items:

1. That throughout the present construction at every point where the trench is in a rock with inclined strata showing open seams or other evidence of actual or threatened disintegration, the sides be shored across the trench so as to form a continuous strut from side to side of the trench, with additional supports in the shape of raker braces if projecting portions of rock afford the necessary hold.

2. That an effort be made to systematize the work so that the sequence of operations will carry the work of excavation and the erection and concreting of the permanent subway structure close together, so as to require as little falsework and temporary decking of the street as possible.

3. That in future work preference be given to a type of timbering that will provide continuous needlebeams or struts from side to side of the trench, with the joints of the uprights firmly spliced to prevent buckling.

4. That a periodic inspection be made of every point of contact of shoring with the banks, as well as of all connections of members, to insure continuous tightness and prevent deterioration of the timberwork from vibration.

They praise the general conduct of the work and the intelligent forethought and faithful execution of plans. Continuing this excellent practice, and observing also the four precautions recommended, the public will be fully safeguarded against repetition of the accidents, they say, in spite of the element of risk inseparable from the work.

Panama Slide Increases

Unofficial dispatches from the Canal Zone state that the slide in Gaillard Cut which started on Sept. 18 is increasing in seriousness and that now nearly 10,000,000 cu.ft. of earth is in motion over an area of 175 acres. It extends 2,600 ft. along both banks and has an average width of about 1,500 ft. back from the canal center line. No official prediction of the date of opening of the canal has been made, but it is stated that practically all the ships waiting for passage have been ordered to other destinations.

This news is confirmed by the following description of the Culebra or Gaillard Cut slides published in the *Canal Record* for Sept. 15, 1915, just after one of the canal stoppages had been cleared up:

The section of channel affected by the two Culebra slides extends between Sta. 1770 and 1798, a distance of 2800 ft. The east slide lies directly to the north of Gold Hill, and the west slide to the north of Contractor's Hill. The width between the outer limits of the breaks on opposite sides of the cut is greatest at Sta. 1787, where it amounts to 2975 ft.

On the east side the break extends back 1525 ft. from the center line of the channel, and the surface at the outer limit of present breaking is 350 ft. above sealevel. On the west side the break runs back 1450 ft. from the center line, and the present edge is at 585 ft. above sea level. The greater part of the movement at this time is from the west side. On July 1, it was estimated that 5,000,000 cu.yd. were in motion on the west bank in this section, and 4,000,000 cu.yd. on the east bank. Since that date about 2,000,000 cu.yd. of spoil has been removed at the bases of the two slides.

In the past month and a half the forces of the Dredging Division have made some remarkable records in excavation. Five hard material dredges have been employed at the bases of the Culebra slides and they and all the accessories serving them have been worked under heavy pressure 24 hr. a day.

During the month of August the excavation at the bases of the Culebra slides amounted to 793,012 cu.yd. In the same month 66,228 cu.yd. was removed from the base of Cucaracha slide, and the seagoing suction dredge Culebra, which ranges the cut between Gamboa and Paraiso to remove silt and keep the channel clean, took out 132,639 cuyd. The total excavation in the cut by seven dredges was 991,879 cu.yd. From September 1 to the morning of September 13, the total excavation from the Culebra slides was 291,082 cu.yd. and the total from the cut was 380,127.

Until it is known how much of the total slide actually threatens the section of the cut, the time required for the clearing of a ship channel in uncertain.

Major-General Goethals, whose resignation as Governor of the Zone was to have taken effect next month, has returned to Panama from the International Engineering Congress at San Francisco and has announced that he would remain at the canal until "its condition would permit his departure," presumably until the slide situation is satisfactory.

News of the Engineering World

Old Wall Used as Coffer-Dam Fails at Louisville

A section of the old wall of the Louisville & Portland Canal, at Louisville, Ky., which is being widened from 85 to 200 ft., failed on the morning of Oct. 5, admitting a rush of water that flooded the new portion of the canal. A hoisting engineer employed by the contractor, the Henry Bickel Co., of Louisville, was drowned. With a companion he was in a flatboat in the canal opposite the destroyed section, when they saw it begin to move. Both jumped for the solid section, and the engineer, missing it, was caught in the suction. Workmen in the cut, warned by distress whistles from a nearby plant where the accident was seen, escaped with a wetting. Investigation was made immediately by Major J. C. Oakes, Corps of Engineers, U. S. A., in charge of the Louisville District.

The wall was built in 1854 and consisted of cut-stone masonry on a ledge of rock, the construction being shown in the accompanying cross-section (Fig. 3). The contractors had channeled a short distance from the back of the wall, leaving the wall itself with a section of the underlying rock as a coffer-dam. A view of the work, with the old canal on the left and the old masonry wall and ledge in the middle, is given in Fig. 1.

A section 500 ft. in length slid along the seam at El. 404.2, a little over 4 ft. from the bottom, and toppled into the cut. Another section of 100 ft. slid along the bottom of the canal a distance of 50 ft. into the cut, but remained upright. Two sand barges that were tied to the wall near the scene of the break were swept through by the rush of the waters and carried against the new canal wall, while a steamboat which was passing was swept against the old wall. The concreting plant inside the cut was pushed aside about 60 ft. and swept off the rails. The condition after the failure is shown in Fig. 2. The location may be tied up to Fig. 1 by noting the power-plant intake on the left, common to the two views.

The excavation had been completed several hundred yards beyond the point of failure and the wall had been

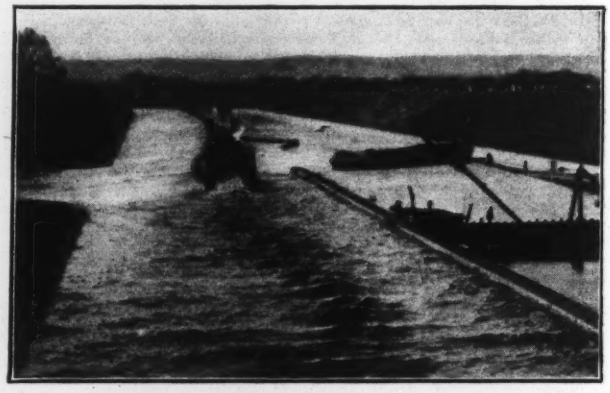


FIG. 2. LOOKING DOWN CANAL TOWARD DESTROYED SECTION OF WALL

standing as it was for several months. The water in the canal had risen about 1 ft. above pool level. It was suggested by a member of the contracting firm that the waves of the passing steamboat had aided in lifting the wall, and by increasing the height of the water added the pressure necessary to move it. Both the Government en-

FIG. 1. WIDENED SECTION OF CANAL BEFORE FAILURE

gineers and the contractor's superintendent, A. G. Butler, had kept a constant watch on the wall and the failure occurred without any previous indication. Butler was on the wall close to the part that failed and was almost caught by the inrush.

The masonry had been reinforced on the excavation side with concrete some time previously. Leaks that had developed had been sealed by applying bags of manure

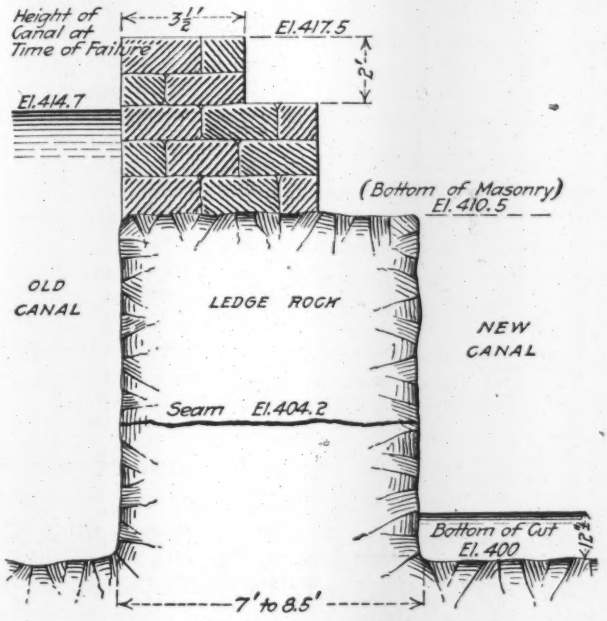


FIG. 3. SECTION THROUGH OLD WALL, LOUISVILLE & PORTLAND CANAL

to them on the canal side and allowing the water to draw them into the holes.

It is probable that the contractor will build a cofferdam beyond the break, permitting the water to remain in the portion already completed. Considerable machinery and trackage were caught, but these can be fished out. The accident is the first that has occurred either in the widening of the canal or the construction of the new locks. A ditch at the lower end of the work will drain a large part of the water after the river falls.

Controlling River des Peres

A 13-mi. river improvement scheme at a cost of $6,000,000 to eliminate flood dangers in the River des Pères valley, St. Louis, has been proposed in a report just made by the Board of Public Service at the request of the Board of Aldermen of that city. The project includes 7 mi. of closed conduits to take the whole of the extreme flood flow and the sanitary sewage of the upper part of that portion of the river within the city and 6 mi. of open storm-water channel to take maximum-flood flows. Beneath this channel there would be a sewer (already built) which would take all the sanitary sewage of the district, the entire dry-weather flow of the river and the storm water from light showers.

The 7 mi. of combined sewer would begin as a single conduit and continue as two and then as three conduits, each 30 ft. in width. The estimated cost of this part of the work is $4,500,000. The lower 6 mi. (open channel) would range from 70 to 100 ft. in width and from 15 to 25 ft. in depth.

The estimated cost of excavating the channel, lining it with masonry and filling the old bed of the river, where property is valuable, is $1,500,000. The existing sewer beneath the proposed open channel cost about $930,000. To complete the system several branch conduits would be required, taking in small streams. The River des Pères drains 16,000 acres within the city and 54,000 outside it in St. Louis County. Serious floods occurred in the valley in March, 1897, October, 1905, both June and July, 1912, March, 1913, and, most serious of all, August, 1915. (For accounts of the last-named see *Engineering News* of Aug. 26 and Oct. 7, 1915.)

Steamship Line Uses Oil Fuel

The Panama-Pacific Line has just announced that oil-fuel will replace coal under the boilers of the twin-screw steamers "Finland" and "Kroonland," now making the 17-day run with passengers and freight between New York, Los Angeles and San Francisco through the Panama Canal. The necessary changes will be made during November and December for installation of mechanical low-pressure oil-burning apparatus (White system). Besides assuring clean decks and eliminating dust, cinders, coaling delays *en route*, etc., the change will allow higher speeds and more continuous high speed. A 40% reduction in fuel bills and a decrease in the fireroom staff from 60 men to 12 are expected.

The "Finland" and "Kroonland" are the largest steamers ever to be equipped in this manner, being 580 ft. long, 60 ft. broad and having a displacement of 22,000 tons. The fuel will be stored in their double bottoms, and this will make available one-third additional cargo capacity, now used for coal bunkering. The same system has worked satisfactorily on the coastwise steamers "Bunker Hill" and "Massachusetts" and in the fleets of the Standard and Texas oil companies.

Structural Damage Slight in New Orleans Storm

The severe windstorms of the past year at Cincinnati, Galveston and New Orleans have all demonstrated that the structure of a well-designed building is safe against the pressure of any wind. In further comment on this observation J. F. Coleman, consulting engineer, of New Orleans, and W. H. P. Creighton, Dean of the College of Engineering at Tulane University, write regarding the Sept. 29 storm at that city when all wind-velocity records were broken:

Mr. Coleman's Comments

Mr. Coleman's comments in part are as follows:

"Practically all of the buildings which collapsed during the storm were light frame structures or very old brick buildings. In the business center of the city the damage consisted principally in broken windows, ripped-off skylights and slate roofs, blown-off chimney tops, and blown-down smokestacks. The tall structural-steel buildings had a very distinct oscillation during the height of the storm, though so far as I know this oscillation was not measured or even estimated. However, it very evidently was not as great as some of the occupants of such buildings

thought, as there is no evidence of cracks in the brick, stone or terra cotta, with which they are faced.

"On the river front the steel wharfsheds suffered to some extent by having the corrugated-iron siding and the wooden sheathing and composition roofing partly stripped off. Some of these sheds appear to have suffered no damage and the only one which collapsed was wrecked as the result of a boat being blown against the wharf on which it rested, so damaging the wharf as to cause wreck of the shed. In the residence districts few houses escaped unscathed, though a number were badly injured.

"In general retrospect it is surprising that the damage was not greatly in excess of what it is. Our modern buildings have been usually designed for wind loads from a 60-mi. velocity, and no doubt in many instances for lesser loads. For 3½ hr. the wind had a velocity in excess of 60 mi. per hour, reaching 86 mi. sustained velocity with one gust of 120 mi. This city contains approximately 75,000 buildings, of which number not much more than 50 were wrecked, and these mostly of the flimsiest frame construction. The casualties within the city were 15 killed and 40 injured."

Professor Creighton's Report

Professor Creighton sends the following information supplementing his report of last week:

"Last week your correspondent declared that no substantial, well-built, modern structure suffered more than trivial damage. After interviewing a number of superintendents, consulting engineers and architects, and after examining a number of wrecked structures, no reason has as yet been discovered for changing that opinion. In the early part of an interview the superintendent or owner would make broad claims for the disabled structure, but ultimately would admit some defect. Incidentally, it must be admitted that it is not necessarily economical to design all buildings to withstand hurricanes.

"The steeple of a prominent church, built in 1857, fell. The mass of wreckage at the foot of this steeple is largely made up of single bricks with no mortar on one side. The failure of the mortar to adhere at all to the brick indicates defective adhesion and probably points to a failure to wet the bricks before laying them. An architect remarked that the mortar on the bricks of overturned house chimneys had evidently deteriorated as it seemed largely composed of sand and that he had noticed no cases where chimneys laid up in cement mortar had fallen.

"New Orleans has now an unexcelled water filtration plant. During the storm the water pressure dropped only 5 lb., although several mains of 8 in. or so were completely broken and caused a great leakage. Formerly, however, people depended upon conducting the rain which fell on the roofs into cisterns made of cypress. This has resulted in the city of New Orleans having more slate roofs than any other city. Slates sweat a little in the early morning before the sun is high and this rusts the iron nail that holds them on the roof. When the head of the nail is rusted badly it requires but little wind to blow the slate away. Fancy tiles should be fastened, preferably by copper fastenings, to the roof or ridge pole. Similarly slates and shingles would be more permanent if fastened with copper fastenings. In this storm a Bedford limestone block about a foot square and three feet long was pushed off a part of the facade of a church. The builders evidently thought that its weight ought to be ample to keep it in place.

"The shipping on the river front was not furnished with heavy enough lines. A couple of light 3-in. hawsers are insufficient to hold vessels of any considerable size. Many of the vessels should have been held by 6-in. lines.

"All observers noticed the considerable pulsations of the barometer during the storm. The wind came in strong gusts. There seems to be no doubt that at times the pressure outside the roof of a building was less than the pressure inside and as a result more or less of the roof would be lifted off. Thus at Tulane University there was a large tank sunk in the ground and covered with boards 3 in. thick, 12 in. wide and 7 or 8 ft. long. The upper surface of these boards was even with the surface of the ground. During the storm the boards were lifted and dropped into the tank. Architects take much care in designing structures against the downward pressure of the air and make little or no provision against the effects of any lifting power it may develop. This cyclone searched out hundreds of open sheds and lifted corrugated-iron roofing and sent it flying, or if the roofs held fast it raised the sheds clear of the ground, and when the sides were well inclined to their proper position, flattened the whole structure."

Wall of Covered Reservoir Slides Out

The clearwater reservoir of the Portsmouth, Ohio, filtration plant was wrecked on Sunday, Oct. 3, by the failure of a 75-ft. length of concrete side wall. The wall at this point was about 30 ft. high and rested on rock of poor quality. The breakout was sudden, although leakage from the embankment outside the wall had been noticed several hours before, and a moist condition of the bank had prevailed for some time previously.

The reservoir is 260 ft. by 260 ft., about 16 ft. deep from floor to roof, with a capacity of about 8 million gallons. The roof is a reinforced-concrete slab on brick piers. The floor is laid directly on the ground (part on fill). The wall is reinforced concrete, 18 in. thick at top, 30 in. thick at floor, and continuing on uniform batter to rock. Buttresses on the water side had their reinforcing tied into longitudinal rods in the wall.

At the point of break the bottom of the wall was about 14 ft. below floor, or 30 ft. below roof, and the reservoir floor was on backfill. The rock contained mud seams, and its surface sloped downhill. The failure appears to have been a sliding on the rock.

Local reports name the J. F. Witmer Co. as engineers for the reservoir. Curtis & Shumway were contractors. Although in use, the reservoir had not been accepted or fully paid for, on account of lack of tightness.

Detroit-Superior Arch Closed—The closing members of the 591-ft. steel arch crossing the Cuyahoga River in the new Detroit-Superior viaduct at Cleveland, Ohio, were placed Oct. 6, 1915. The middle lower chord members of both trusses were placed simultaneously.

A Company of Engineers has been organized in the cadet brigade at the University of Illinois. There will be a captain, two lieutenants and 54 men. The company will receive some infantry drill, such as marching and company movements. There will be also "sand-table" lectures on trench construction and earthwork and possibly topographic surveying and map sketching. It is expected to develop men who would be valuable in the organization of volunteer engineer companies. A similar organization has already been noted in the Cornell University Cadet-Corps.

Cast-Iron Dome for Sale—The old cast-iron dome of the capitol at Madison, Wis., has been taken down to make way for the new dome and is now for sale. The various members were removed in such form as to permit their reërection and are now stored on the university campus at Madison ready for the purchaser. All pieces are marked to fit a specially prepared erection plan to facilitate erection. The total weight of metal is estimated at between 800 and 1,000 tons.

PERSONALS

Mr. J. B. Holtzworth has been appointed Assistant Engineer of the Western Maryland Ry., to succeed J. M. Harris, who has entered the employ of the Poole Engineering Co., of Baltimore, Md.

Mr. W. H. Gould, formerly Sanitary Engineer of Lucas County, Ohio, will in the near future open an office in The Nasby, Toledo, where he will engage in the practice of civil and sanitary engineering.

Mr. Joseph Fawcett, formerly Surveyor of Greene County, Ohio, has been appointed County Road Superintendent under the new Cass law, which places the county surveyor at the head of the road department.

Mr. G. H. Hutchinson, well known in the Middle and Far West among contractors, has become associated with the Bucyrus Co., of South Milwaukee, Wis. He will be in charge of its recently opened office at 900 Hibernian Building, Los Angeles, Calif.

Mr. A. C. Watson, Division Engineer of the Logansport division of the Pennsylvania System, has been appointed Division Engineer of the Cleveland & Pittsburgh division. Mr. Watson will fill the vacancy created by the recent death of E. F. McCrea.

Mr. F. W. Brown, formerly Superintendent of the Louisville division of the Southern Ry., has been promoted to the position of Assistant to the General Manager, with headquarters at Washington, D. C. He will have general charge of train tonnage and efficiency.

Mr. Louis A. Boulay, formerly associated with the firm of the Smith & Boulay Co., Consulting Engineers, of Toledo, Ohio, and now of the recently organized firm of L. A. Boulay Co., Civil and Sanitary Engineers, 1248 Nicholas Building, Toledo, has been appointed Sanitary Engineer of Lucas County, to succeed W. H. Gould.

Mr. Arthur C. Eversham, M. Am. Soc. C. E., formerly Assistant Chief Engineer of the Kansas City Terminal Ry., is now associated with the Kansas City Bridge Co. as Engineer of Construction. He will have entire charge of the construction work of the company and will give special attention to government and railroad construction.

Sir William H. Lindley, City Engineer of Frankfort, Germany, from 1884 to 1896, and engineer for water-works and sewerage improvements in that and many other German cities, has been honored with the degree of Doctor of Engineering by the Technical High School of Darmstadt, Germany. His father, William Lindley, designed the sewerage system of Hamburg after the great fire, which was the first modern sewerage system in Germany.

Messrs. C. H. Vaughan, M. Am. Soc. C. E., formerly Second Vice-President and Chief Engineer of the Penn Bridge Co., Beaver Falls, Penn., and H. R. Hortenstine, M. Am. Soc. C. E., formerly Contracting Manager of the same company and J. O. Childers, formerly with the erection department of the McClintic-Marshall Construction Co., and in charge of the erection of the lock gates at Panama, have purchased the plant of the York Bridge Co., of York, Penn., which will be incorporated under the name of the York Bridge and Construction Co.

OBITUARY

Axel Samuel Berquist, M. Am. Soc. C. E., inventor and patentee of the suspension bunker which bears his name, died at his home in Brooklyn, N. Y., on Oct. 6, after a long illness.

William M. Whelan, Superintendent of the Elizabethtown Water Co., of Elizabeth, N. J., died on Oct. 3 at the home of his sister in that city, at 58 years of age. Mr. Whelan had been in the employ of the water company since boyhood.

L. A. Fowler, head of the contracting firm of L. A. Fowler & Son, died recently at his home in Kansas City, Mo. This firm has done a large amount of building work in Kansas City. One of its larger contracts was for the annex to the Baltimore Hotel in that city.

Arthur S. Mann, M. Am. Soc. M. E., Mechanical Engineer of the General Electric Co., of Schenectady, N. Y., and formerly lecturer at Columbia University, died at Saranac, N. Y., on Oct. 3, at 45 years of age. He was born at Medbury, Mass. Mr. Mann was an authority on steam generation. At various times he also was associated with the Allis-Chalmers Co. and the Metropolitan Street Ry. Co., of New York City.

Gustave C. Kuhlman, founder and former President of the G. C. Kuhlman Car Co., died on Oct. 4 at his home in Cleveland, Ohio. He had been in ill health for some time and had recently suffered a nervous breakdown. Mr. Kuhlman was one of the pioneers in the street-car building industry in the Middle West. In 1881 he organized the company which still bears his name and opened a shop on St. Clair Ave., Cleveland, where he built horse-drawn and cable cars. When electricity was introduced for street-car systems the plant was expanded and moved to the Broadway car barns at Aetna Road. In 1903 Mr. Kuhlman sold his holdings in the company to the J. G. Brill Co., of Philadelphia, at which time the Kuhlman Co. was reorganized. Since his retirement from the company Mr. Kuhlman had been northern representative of the Martindale Mercantile Agency. He was 56 years of age.

ENGINEERING SOCIETIES

COMING MEETINGS.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 19-21. Annual meeting, at Detroit, Mich. Secretary, C. A. Lichty, 319 N. Waller Ave., Chicago.

ASSOCIATION OF RAILWAY ELECTRICAL ENGINEERS.
Oct. 18-24. Annual meeting in Chicago. Secretary, J. A. Andrencetti, C. & N. W. Ry., Chicago, Ill.

MAINTENANCE OF WAY AND MASTER PAINTERS' ASSOCIATION.
Oct. 19-21. Annual meeting in St. Louis. Secretary, T. I. Goodwin, C. R. I. & P. R.R., Eldon, Mo.

NATIONAL MUNICIPAL LEAGUE.
Nov. 17-19. Annual meeting in Dayton, Ohio. Secretary, C. R. Woodruff, North American Bldg., Philadelphia, Penn.

Southwestern Electrical and Gas Association—The 12th Annual Convention will be held at Galveston, Tex., May 17-20, 1916. H. S. Cooper, the secretary, recently made a trip to Galveston to see whether the storm had rendered it inadvisable to hold the convention there. He states that he found the business of the city almost normal and that there is no reason for changing the place of meeting.

American Institute of Chemical Engineers—The winter meeting will be held in Baltimore, Md., Jan. 12-15. The date selected is later than usual owing to the summer meeting having been held late in August in San Francisco. As the attendance of Eastern members at the San Francisco meeting was small, it is hoped that they will turn out strongly for the Baltimore meeting. The secretary is J. C. Olsen, Cooper Union, New York City.

National Lumber Manufacturers' Association—A series of meetings has been planned by the trade extension department of the National Lumber Manufacturers' Association, which is designed to teach the general public the advantages and general availabilities of wood as a building material. The first of these meetings was held on Sept. 23 in the rooms of the Commerce Club, Toledo, Ohio. The secretary of the association is R. S. Kellogg; the manager is E. A. Sterling, 925 Lumber Exchange, 11 South La Salle St., St. Louis, Mo.

Philadelphia Association of Members of the American Society of Civil Engineers—The annual meeting of the Philadelphia Association was held at the Engineers' Club, of Philadelphia, on Oct. 4. Dr. John A. Brashear, president of the American Society of Mechanical Engineers, delivered an illustrated lecture on "Contributions of Photography to our Knowledge of the Stellar Universe." The officers elected were: President, Edward B. Temple; vice-presidents, Edgar Marburg and John Sterling Deans; directors, J. W. Ledoux, Henry H. Quimby, H. S. Smith and George A. Zinn; treasurer, S. M. Swaab; secretary, W. L. Stevenson.

International Gas Congress—A congress representing the American gas industry was held at San Francisco from Sept. 27 to Oct. 2. The formal sessions opened with the presidential address of A. C. Humphreys, in which he pointed out that public utilities have to face a condition of over-regulation by public authorities. He believed that all should be active to

correct the misleading statements constantly being made. While admitting occasional faults, he claimed that generally the gas business is carried on with due respect to the rights of the public. The first technical paper, by O. B. Evans, was a historical review of the development of water gas. A. G. Glasgow followed with a paper on deep fuel combustion in water-gas manufacture. R. S. McBride, of the United States Bureau of Standards, then presented a paper on the substitution of heating value for candlepower as a criterion of gas quality. Many state and city authorities have already acted to establish the heating-value standard. In the discussion it was suggested that 600 B.t.u. per cu.ft. would probably be generally accepted as the standard. On the second day the congress met in two sections. One considered half a dozen papers on gas for illumination; the other took up nine papers on commercial aspects of the gas business. The discussion in both sections turned largely on the competition between electricity and gas for illumination purposes. The majority of those present represented companies supplying gas alone, and they declared that gas could hold its own in competition with electricity. In order to hold their business in the face of low electric rates the gas companies have engaged in a systematic campaign with both owners and architects, even in some cases furnishing free labor for installing the gas pipes. On the third day a review of modern coal-gas processes was given by the Society of Gas Engineering of New York. A general movement was noted in the East to substitute coal gas for water gas on account of the increase in the price of oil and the uncertainty regarding the future supply.

Appliances and Materials

Flow Meter for V-Notch Weirs

A flow recorder to be used in connection with small V-notch weirs has been designed by the engineers of the Harrison Safety Boiler Works and will be used particularly for measuring-type boiler-feed water heaters. There is a float which moves with each change of head on the weir and in turn rotates a spiral cam (laid out according to the law of the weir, $CH^{\frac{5}{2}} = Q$) through a multiplying gear. The cam moves a pen carriage which records on a paper drum the quantities corresponding to each position of the float. Back-lash has been eliminated by the use of a cable drum instead of a gear and pinion, while friction has been reduced as much as possible by mounting the spindle of the cam upon antifriction rollers. The pen carriage is provided with large rollers or wheels which rest upon horizontal ways, so that the cam follower which is attached to the pen carriage moves diametrically to the cam disk. The pen is suspended from the pen carriage so that it rests lightly against the chart drum and can readily be removed for cleaning or renewing the chart. An integrating attachment consists of a counting train, likewise suspended from the pen carriage and driven by a small roller, which rests upon an aluminum clock-driven disk. When the float and cam are at the position corresponding to the zero head, this roller is at the center of the disk, and hence receives no motion from the rotation of the latter. As the pen carriage is moved away from the zero position, the small roller is carried away the same distance from the center of the aluminum disk, and hence is propelled at a rate corresponding to the rate of flow over the weir. The total movement of the counting train will therefore correspond to the total flow. Further, a visible pointer moving along a large scale with open divisions has been added, making it possible to read the rate of flow from a distance.

HARRISON WEIR FLOW RECORDER

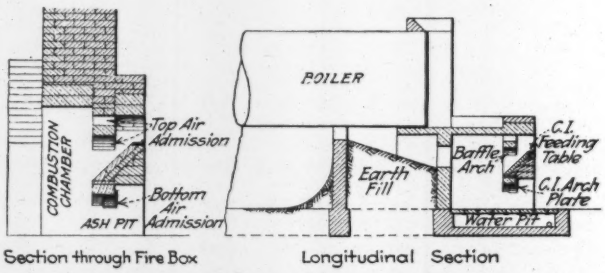
All parts of the mechanism of this recorder are made to accurate dimensions by means of templets and limit gages. To be sure of the accuracy of the instrument it is only necessary to know that it is properly adjusted—that is, when the float is at zero position; in other words, when the level of water upon which it rests is at the zero level of the weir, the cam must be at zero position, the pen on the zero line of the chart and the integrator roller at the center of the aluminum disk.

* * *

Smokeless Firebox for Kilns and Boilers

A radically new design of furnace for clay-products kilns and boilers has been devised by Paul Beer, President of the Grateless Furnace Co., Des Moines, Iowa. The arrangement is shown in the accompanying sketches. A rectangular firebox has a sloping fuel table leading down from a firing hole and having an ignition arch thrown over it. Beneath is an arch and plate spanning the ashpit. The combustion chamber is forward of the coking table and is spanned by a high arch.

The most difficult requirements of clay-products kilns have been satisfactorily met, it is reported by A. W. Clement, Smoke Inspector of Des Moines. During the early firing of a kiln, when low heat is required with great excess of air, the fuel bed fills the space below the bottom of the coking table. The upper portion of the firebox remains open and unobstructed. A little air is supplied through the ashpit. For the second period, which requires an oxidizing heat, coal is piled part way up the coking table. As the fires need cleaning ash and clinker are removed from underneath the coking plate edge and fresh coked fuel falls down to replace it. Volatile matter passes over the incandescent bed of coked fuel below. More air is supplied from the front of the ashpit, which is entirely open and without grate bars. When excess



GRATELESS GRAVITY-FEED KILN AND BOILER FURNACE

air is required the upper part of the firebox is left partly open. During the high heat for vitrification the fuel bed is carried higher up to close the fire hole and all air is cut off over the fire except that which comes through a small hole in the baffle arch. Air admission through the lower fuel bed is obtained by regulating the cleaning period. A reducing gas can be produced without smoke. A saving of nearly $60 per kiln per heat is reported in the paving-brick plant of the Barbour Asphalt Paving Co. The time of firing was reduced from 11 days 8.7 hr. to 9 days 14.3 hr.

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Notary Public.
(My commission expires March 30, 1917.)